CHANNEL REDESIGN: FLOOD MITIGATION FOR THE UNIVERSITY OF NORTH CAROLINA AT CHAPEL HILL COKER ARBORETUM DRAINAGE CHANNEL

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A Technical report submitted to the Faculty at the University of North Carolina at Chapel Hill in partial fulfillment of the requirements for the degree of Master of Science in Environmental Engineering in the Department of Environmental Sciences and Engineering in the Gillings School of Global Public Health

Chapel Hill
2015

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ABSTRACT

Jesse Randall Phillips: Channel Redesign: Flood Mitigation for the University of North Carolina at Chapel Hill Coker Arboretum Drainage Channel
(Under the direction of Pete Kolsky)

The Coker Arboretum drainage channel is prone to flooding during heavy storm events, such as the storm event that occurred on June 30th, 2013. The flooding on June 30th, 2013 caused about $4,200 in damages to the arboretum walking paths and sent large amounts of sediment-laden stormwater into Raleigh Street to the East. This report focuses on channel redesign as a means for flood mitigation in the Coker Arboretum. Hydraulic and hydrologic modeling, technical consultations, and field investigations were used to explore five channel redesign options under two main approaches, peak flow attenuation and an increase in channel discharge capacity. Dry detention basin performance was analyzed in an attempt to achieve peak flow attenuation. For an increase in discharge capacity, the channel was redesigned such that water levels did not surpass a critical depth, including freeboard, during a 10 year SCS – Type II design storm. The most functional and cost effective solution was determined to be an increase in discharge capacity. An implementation plan was developed and project costs were compared to the present value of future benefits.

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LIST OF ABREVIATIONS

CFS	Cubic Feet per Second
CHI	Computational Hydraulics International
EHS	Environmental Health and Safety
EPA	Environmental Protection Agency
ESD	Energy Services Department
ESD	Energy Services Department
HEC-RAS	Hydrologic Engineering Center River Analysis System
HGL	Hydraulic Grade Line
NOAA	National Oceanic and Atmospheric Administration
NPV	Net Present Value
OWASA	Orange Water and Sewer Authority
PCSWMM	Personal Computer StormWater Management Model
SCS	Soil Conservation Service
SWMM	StormWater Management Model
T_c	Time of Concentration
UNCCH	University of North Carolina at Chapel Hill

CHAPTER 1: INTRODUCTION

The Coker Arboretum and botanical garden is located on the northern side of the University of North Carolina at Chapel Hill (UNCCH) main campus, between Franklin Street and East Cameron Avenue and is bordered on the east by Raleigh Street. The Arboretum is managed by the North Carolina Botanical Garden and is one of the Garden's oldest tracts; it was created in 1903 by Dr. William Chambers Coker and now contains hundreds of native plant species. The community greatly values the Coker Arboretum and it is considered a very high quality environment ("Coker Arboretum", 2014).

An open channel drains stormwater runoff from the arboretum and immediate surroundings, as well as a number of upstream subcatchments which drain into the upstream end of the channel. This drainage channel has been subjected to flooding during heavy storm events, resulting in damaged walking trails and conveyance of sediment-laden stormwater onto Raleigh Street to the east. The arboretum and the UNC Energy Services Department (ESD) is considering a number of solutions to assuage drainage channel flooding.

This report represents the synthesis of three technical briefs that sought to: (1) identify the nature and cause of the drainage channel flooding problem; (2) explore a number of technical solutions focusing on channel redesign and select the recommended solution; and (3) create a plan for implementing the chosen solution.

Chapter 2 discusses the nature and identifies the likely causes of arboretum flooding. Upstream stormwater infrastructure and drainage characteristics are reviewed along with relevant channel characteristics. The most problematic sections of channel are identified and the impacts of flooding are discussed. Chapter 2 also relates proposed and applied stormwater control strategies and their effect on channel flooding.

Chapter 3 proposes a number of technical solutions to alleviate flooding. Each option is then designed to a conceptual level and analyzed for its effect on drainage control. Comparisons are drawn between the proposed solutions under a number of metrics in Chapter 4, most importantly effective flood mitigation and planning level costs.

A detailed implementation overview is given in Chapter 5. This includes a description of the review and approval process, a construction outline, a review of scheduling and disruption, and a more detailed estimation of costs. Chapter 6 presents a cost benefit analysis that compares estimated capital costs to the present value of future benefits and the net present value of the project is determined.

CHAPTER 2: PROBLEM IDENTIFICATION

Introduction

This chapter explores the nature of the Coker Arboretum drainage channel flooding problem. Drainage area and surrounding infrastructure characteristics are discussed in an attempt to define the causes and impacts of flooding during heavy storm events. Susceptible areas of concern are described and an overview of current and historical flood mitigation practices and proposals is given. Furthermore, characteristics of the channel and surrounding landscape are analyzed to describe their relationship to channel flooding.

Drainage Description

The Coker Arboretum drainage channel has a total drainage area of about nine acres (see Table 1 below) and is in the Battle Branch watershed, which totals around 670 acres. Battle Branch is closely bordered on the west and north by the Mill Race Branch watershed and on the west and south by the Meeting of the Waters watershed. The aforementioned watersheds are highly impervious and contain many of the older buildings and brick walkways on campus. This has been noted to exacerbate surface flow and flooding issues by UNC staff who have conducted field visits (Hoyt, 2014; MacIntyre, 2014).

A stormwater infrastructure and watershed map of the Coker Arboretum and immediate surroundings is shown in Figure 1. Figure 1 was provided by the UNC Energy Services Department (ESD) and uses data from the ESD GIS database with permission from Lisa Huggins (2014), the GIS coordinator for the UNC Energy Services Department. Please note that the scale is slightly altered due to resizing. Figure 2 shows relevant features of the project site to be discussed throughout this report, with subcatchments delineated by Rummel, Klepper, & Kahl LLP consulting engineers (RK&K) and Biohabitats Inc. in 2013 using the UNC Stormwater Geodatabase as a starting point and incorporating field work as well as other topographical and GIS data. As shown by Figures 1 and 2 below, in addition to the immediate capture of overland runoff, several conduits west of the Arboretum convey stormwater from a number of upstream subcatchments to a 12" pipe and a 15" pipe (conduits 14628 and 11862 respectively) that converge in the open, concrete and stone lined channel in the northwest sector of the Arboretum. The Contributing subcatchments are bordered in red in Figure 2 and relevant catchment information can be found in Table 1 below (Note that averages are weighted according to subcatchment area). The open channel traverses the Arboretum from west to east and drains into a 30" pipe that conveys water under Raleigh Street and into the grander campus pipe network. The open, concrete and stone lined channel that traverses the northern section of the Coker Arboretum ultimately receives much of the stormwater from the surrounding area and will be the focus of this report.

Subcatchment	Area (acres)	% Impervious	Slope (ft/ft)
BATTLE-18	3.6	43	0.043
BATTLE-19	1.4	60	0.041
BATTLE-20	1.6	60	0.055
BATTLE-21	2.3	43	0.035
	Total = 8.9	**Average = 49**	**Average = .043**

Table 1: Relevant Information for Subcatchments Contributing Stormwater Runoff to the Coker Arboretum Drainage Channel

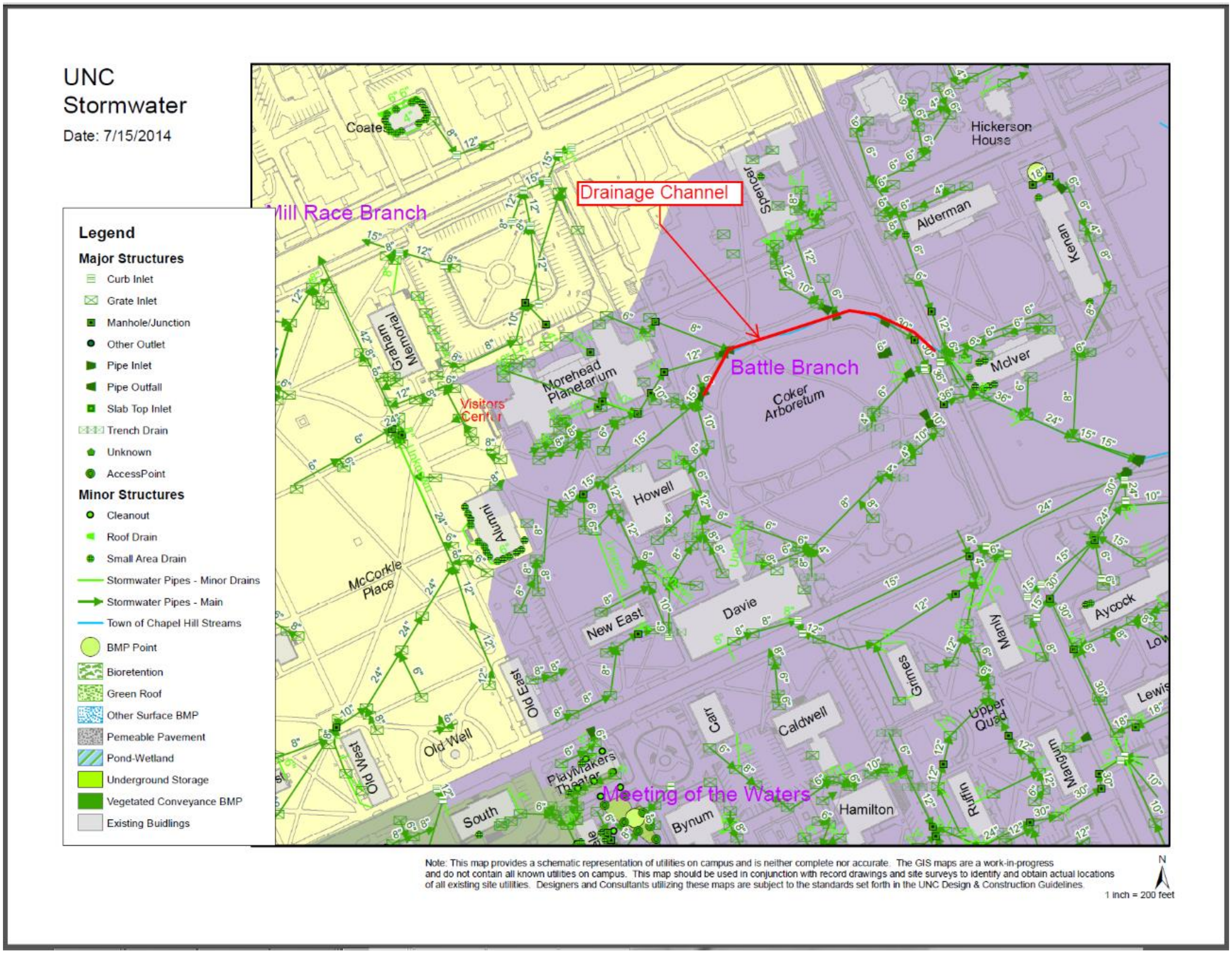

Figure 1: Stormwater Infrastructure Map (UNC ESD, 2014)

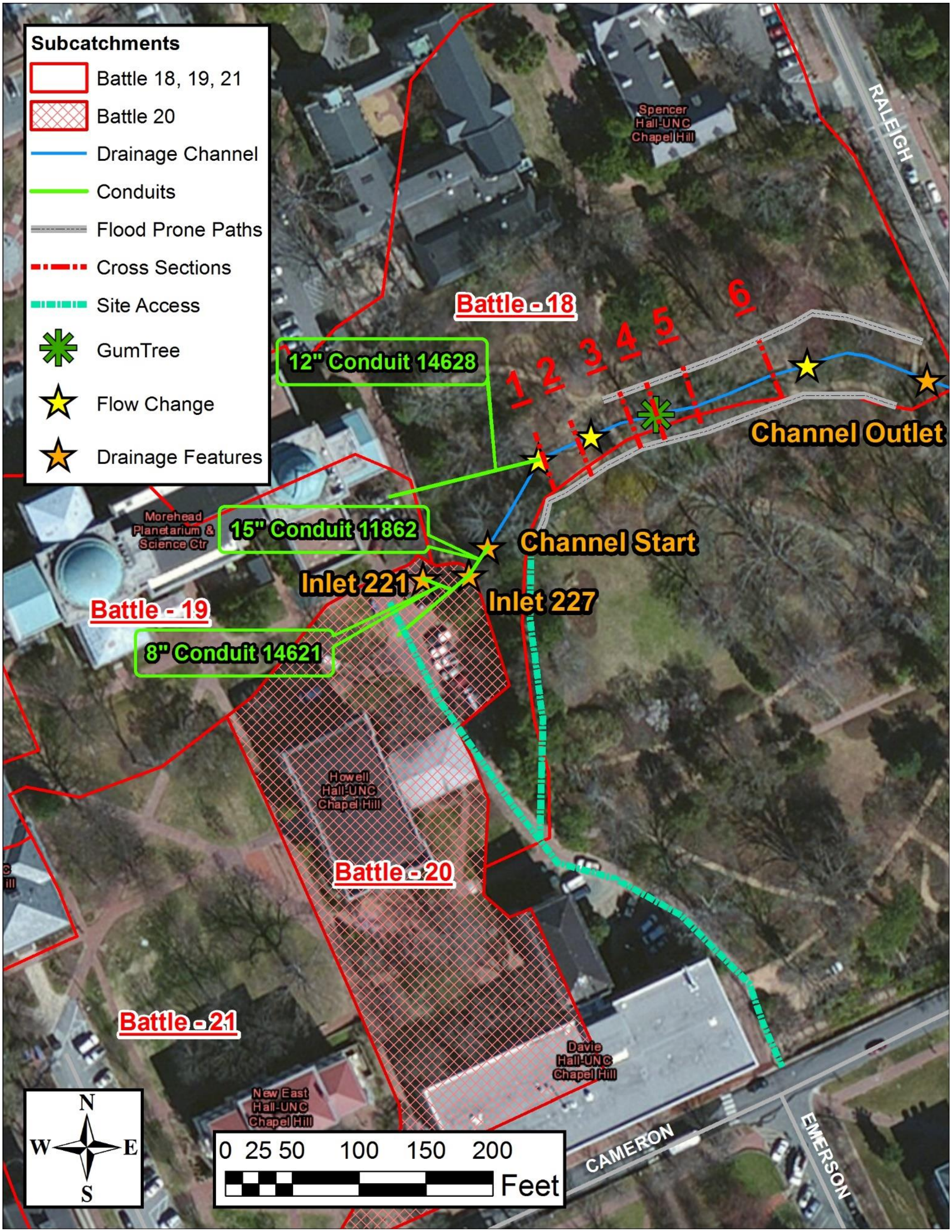

Figure 2: Relevant Features of the Project Site

Areas of Concern

Figure 3 shows the areas of highest flood concern in the Coker Arboretum as outlined by Margo MacIntyre, the curator of the Coker Arboretum who is ultimately responsible for the arboretum grounds and has conducted numerous site visits during rain events.

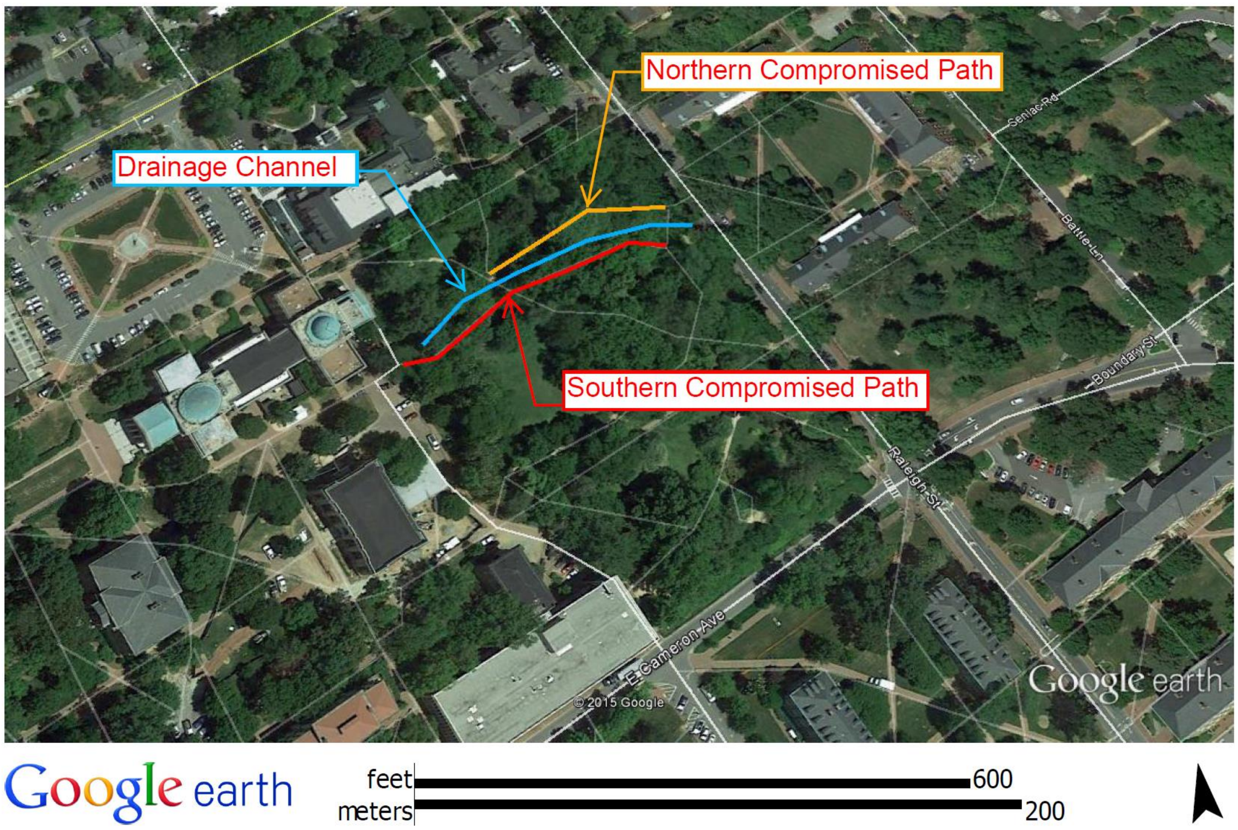

Figure 3: Coker Arboretum Problem Flooding Areas (Google Earth)

Of highest concern is the entirety of the walking path, highlighted in red in Figure 3 above, which enters the arboretum by the southeast corner of the Morehead building and parallels the drainage channel to the south. The path to the north of the channel, highlighted in orange, also suffers from heavy rain events. Both walkways are subject to floodwaters caused by the channel's banks being overtopped, while the southern path receives additional floodwater resulting from upstream stormwater infrastructure issues, discussed in the following section. Figures 4 and 5 below are photos taken by Margo MacIntyre during a significant rain event on June 30th, 2013 and illustrate the extent of flooding experienced by these walkways.

Figure 4: Flooding of Southern Path (MacIntyre)

Figure 5: Flooding of Northern Path (MacIntyre)

The walkways are constructed with crush and run and Chapel Hill grit, both of which consist of small particles and are highly compactable. However, under heavy flooding both surface and base layers are eroded and conveyed in the runoff to Raleigh St., causing the need for extensive repairs. The effects of the walkway sediment transport are shown in Figure 6, another photo taken by Margo MacIntyre during the June 30th, 2013 storm event. Raw material costs, at approximately $25-35 per delivery of a five cubic yard load, are less of an issue than the significant labor costs associated with reparations. With limited equipment access capability, material must be transported by wheelbarrow and spread by hand, which takes an estimated 2-3 person days per path according to arboretum staff. In addition to walkway erosion, flooding can cause habitat destruction and further strain arboretum staff.

Figure 6: Sediment in Raleigh Street (MacIntyre)

Upstream Causes of Flooding

In order to comprehensively describe the nature and causes of the Coker Arboretum flooding problem, stormwater systems upstream of the Coker Arboretum were investigated as well as channel flooding and design. The area upstream of the arboretum ranges from about 40-60% impervious, depending on the subcatchment, with a weighted average of about 49% imperviousness, and includes many of the older buildings on campus and a network of brick pathways. As previously discussed, a number of the highly impervious subcatchments upstream of the arboretum contain stormwater infrastructure that conveys runoff into the upstream end of the arboretum drainage channel. Furthermore, most brick walkways feature a slightly raised border on either edge that prevents flow from leaving the path and entering inlets before reaching the arboretum area. The flows are then concentrated towards an irrigation pump house immediately west of the arboretum. The pump house is surrounded by a stone wall and is served by an inlet (inlet 221) that utilizes an eight inch pipe (conduit 14621 in Figure 2) to convey water to the main 15” pipe.

However, during heavy storm events, the concentrated flows quickly clog the inlet with debris, causing flood waters that collect at the wall to damage vehicles in the adjacent parking lot and to eventually overtop the wall, as shown in Figure 7, and subsequently exacerbate flooding issues for arboretum walkways.

Figure 7: Surcharging of Inlet 221 Adjacent to Pump House (MacIntyre)
(Note overtopping of wall, and water level around car tire in top center of photo)

Furthermore, in order to achieve optimal flow rates, the main 15" pipe is in need of cleaning and repairs in the section that contains the junction with the eight inch pipe serving inlet 221, so surcharging would likely occur to some extent even if the inlet was not clogged (RK&K and Biohabitats Inc, 2014). To further complicate matters, hydraulic grade line profiles completed by Rummel, Klepper, and Kahl, LLP consulting engineers in 2013 suggest that the 15" pipe (conduit 11862) that conveys water to the channel is undersized and the size should be increased to reduce upstream flooding. A properly sized and maintained conduit 11862 at the downstream end of inlet 221 would alleviate some overland walkway flood pressure, especially at the western end of the arboretum where the channel is less prone to overtop its banks. However, the issues described above ultimately prevent much of the runoff from subcatchment Battle – 20 (shown with hatching in Figure 2) from reaching the channel, effectively removing up to 18% of the channel's drainage area. Increasing the drainage area of the channel would result in increased stormwater volume, flow rate, and thus depth, likely worsening flood conditions in the channel section that is already prone to overtop its banks during heavy storm events and erode walking trails, as seen in Figure 8 below.

Current and Historical Flood Control Proposals and Applied Strategies

Measures are being taken to reduce surcharging of inlet 221 shown above in Figure 7 and to better direct flow to appropriate inlets. For instance, the area around inlet 227, just downstream from inlet 221, was recently re-graded and fitted with hardscape improvements to more effectively capture floodwater and runoff to be conveyed into the channel before it reaches the path system. Additional proposed flood mitigation measures include increasing the size of conduit 11862 from 15" to 24" to reduce upstream flooding and altering the construction of brick pathways and re-grading in order to direct flow to swales and inlets.

Furthermore, measures are being taken within the arboretum to reduce the impact of flooding. In order to reduce the propensity for walkway erosion, the arboretum staff employs mechanical compaction and a fairly expensive, relative to raw material costs, chemical stabilizer additive, with mixed results. The arboretum staff has also installed water bars and lateral or perpendicular trench drains in order to divert water to the central lawn area, to more stable paths, or into the stormwater infrastructure system and thus into the drainage channel. However, in instances of heavy rain events, trench drains are clogged and water bars are overtopped, rendering them somewhat ineffective. Lastly, the major outlet that conveys water from the open channel in the arboretum under Raleigh Street to the east and into the larger campus pipe network has recently been updated to a system of 30” and 36” pipes to accommodate higher flows. If all of these flood control measures are effective there will be less direct flood damage on the western sections of arboretum walkways from upstream sources. However, similarly to the aforementioned inlet 221, the results will ultimately serve to direct more flow into the open drainage channel, which already tends to overtop and convey floodwaters into the path system, as shown in Figure 8.

Figure 8: Channel Flooding (MacIntyre)

Channel Flooding

Most recently, the drainage channel overtopped its banks and flooded walkways during the storm event on June 30, 2013. Flooding has been witnessed to be most prevalent in the section of the channel on either side of the westernmost footbridge. The following section describes the channel flooding that occurred on June 30, 2013. Real-time rainfall data from the 6/30/2013 arboretum flood event was acquired by ESD from NC State CRONOS system weather station KIGX at Horace Williams Airport approximately 1.6 miles NNW of the arboretum. The ESD determined that the storm recurrence interval (24 hr. duration) was 10 years by comparing real-time rainfall data with precipitation frequency estimates from the National Oceanic and Atmospheric Administration (NOAA) National Weather Service Data. This information was provided by Sally Hoyt of UNC ESD in September, 2014. The NOAA estimates used data collected from NOAA Atlas 14 weather station Chapel Hill 2 W located at the Orange Water and Sewer Authority (OWASA) on Jones Ferry Road, approximately 1.8 miles WSW of the arboretum. The real-time rainfall data from 6/30/2013 and the NOAA precipitation frequency estimates for a wide range of frequencies and durations can be found in Appendix A. The precipitation frequency estimates are also shown as Intensity-Duration-Frequency (IDF) curves in Appendix A to help visualize the information.

Table 2 shows relevant information, including time of concentration (t_c), for catchments that convey stormwater runoff to the Coker Arboretum drainage channel. Time of concentration was calculated using the kinematic wave formulation according to the StormWater Management Model (SWMM) user's manual, which takes into account, among other parameters, catchment slope, imperviousness, and rainfall intensity. The NOAA estimate for the 10 year 24 hour duration storm was used to calculate catchment time of concentration. T_c calculations can be found in further detail in Appendix B. Note that all averages are weighted according to subcatchment area and that subcatchments BATTLE-20 and BATTLE-21 are in line with one another, so cumulative parameters are also shown. The cumulative t_c represents the time it takes for runoff from the farthest point of Battle – 21 to travel through Battle – 20 and reach the channel at roughly the location of inlet 227.

Subcatchment	Area (acres)	Cum. Area (acres)	% Impervious	Slope (ft/ft)	t_c (min)	Cum. t_c (min)
BATTLE-18	3.6		43	0.043	34.2	
BATTLE-19	1.4		60	0.041	6.6	
BATTLE-20	1.6		60	0.055	6.3	
BATTLE-21	2.3	3.9	43	0.035	10.7	17.0
	Total = 8.9		**Average = 49**	**Average = .043**	**Average = 18.8**	

Table 2: Time of Concentration during the 10 Year 24 Hour Duration Storm for Catchments that Convey Stormwater to the Coker Arboretum Drainage Channel

Along with Scott Rodgers of UNC Engineering Information Services, the author of this report conducted a topographic survey on October 10, 2014 of the arboretum channel section of concern (NW section of arboretum in between the two footbridges) and the surrounding area. Using data gained from the survey, average channel depth for the critical area was calculated by averaging elevation differences across the channel. The average channel depth was compared to SWMM modeling conducted by RK&K Engineers in 2013 that produced a 10-year 24-hour storm hydraulic grade line (HGL) profile for the arboretum channel. The open channel HGL profile describes water surface levels under storm conditions with 10 yr. recurrence intervals. The comparison of channel depth and elevation with the 10 yr. HGL profile concluded that banks would be flooded by anywhere from 0.4 to 0.7 ft. (approximately 5" – 9"), depending on location, during a 10 yr. 24 hr. storm event. Flooding of banks was shown to be greatest in the critical area. This analysis agrees with field reports conducted by Margo MacIntyre, who reported that "Water flow in places was at least six inches deep."

Channel Characteristics

The following section analyzes various channel characteristics as potential contributors to flooding problems. Channel design is an important factor in determining the cause of flooding. As described by the Manning Equation, the effective fall or grade of a channel is important in determining its flow velocity, which in turn is a factor for determining steady state discharge capacity or hydraulic capacity. The grade affects water velocity and thus overall discharge rates. Using data from the October 10, 2014 field survey, it was calculated that the channel has a 1.4% slope in the area of concern. Comparatively, upstream and downstream sections of the channel are characterized by slopes ranging from 2.2% to 2.5%. The grade of the channel decreases by at least 36% and as much as 45% in the compromised area when compared to the rest of the channel.

This can cause the water velocity to decrease, thus decreasing discharge capacity. With all other parameters assumed to be uniform, the Manning equation implies that the slope change alone will decrease water velocity by anywhere from 20 – 25% when compared to upstream and downstream sections. Water subsequently backs up at the critical section and overtops channel banks. In addition to water velocity, the cross-sectional area of the channel is also necessary to calculate steady state discharge capacity by way of the continuity equation. However, this information could not accurately be obtained from the survey described above.

Channel Geometry

Channel geometry field measurements were taken in March 2015 to better assess the existing conditions of the drainage channel in the area of concern. A total of six cross-sections were measured using a measuring tape and a digital level. Cross-section locations and nominal numbering can be seen in Figure 9 below. Because of the limited availability of survey capacity and the fact that channel bed and banks are fairly regular, channel geometry was idealized as regular shapes, as seen in Figure 10.

Figure 9: Channel Cross-Section Locations

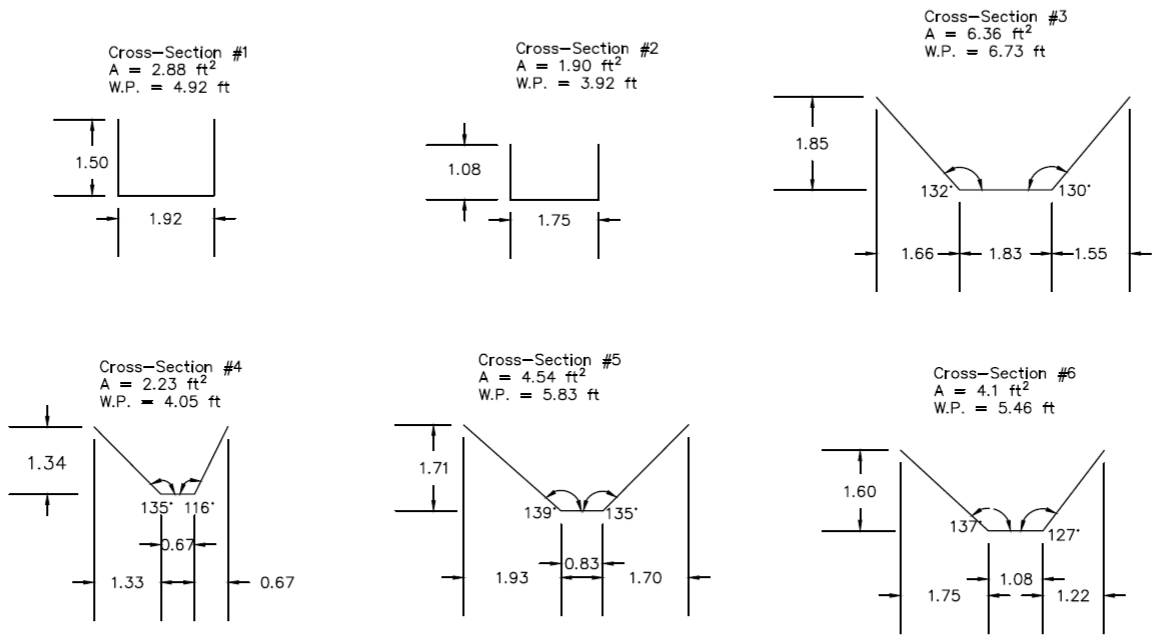

Figure 10: Existing Conditions Cross-Sectional Geometry

The distinct change in channel geometry at cross-section #4 is caused by the roots of a large sweet gum tree invading the channel, further reducing discharge capacity and thus exacerbating channel flooding. Based on the above field measurements and survey data, the Manning and continuity equations were used to calculate the steady state discharge capacity of each cross-section, shown in Table 3 along with the 10 year 24 hour duration design storm peak flows for the respective channel sections. Calculations can be found in Appendix B. A 10 year recurrence interval and a 24 hour duration was selected in accordance with the Town of Chapel Hill Design Manual (2004) requirements for open channel storm drainage infrastructure. Chapter 3 contains a detailed description of the hydraulic and hydrologic modeling used to calculate peak flow rates.

Cross-Section #	Discharge Capacity (cfs)	Peak Flow (cfs)
1	13	27
2	7.4	27
3	33	32
4	8.1	32
5	21	32
6	18	32

Table 3: Comparison of Discharge Capacities and Peak Flow Rates of Cross-Sections

Although cross-sections 1 and 2 upstream of the footbridge do not have sufficient capacity to handle 10 year peak flow rates, the figures in Table 3 take into account only the channel itself. However, that section of the channel features an operationally contained small-scale floodplain that is designed to inundate during heavy storm events. Therefore, further analysis will be limited to the section of channel downstream of the footbridge, represented by cross-sections 3 – 6. As shown in Table 3, the limiting discharge capacity for the section of main concern was calculated to be 8.1 cubic feet per second (cfs). Assuming all stormwater is directed to the drainage channel by the stormwater infrastructure system, the SWMM data estimate that the peak flow rate during a 10 yr. storm event is approximately 32 cfs throughout the channel section of concern. Only one of the cross-sections measured has sufficient capacity for a 10 year storm, neglecting any freeboard that may be required. Even if all stormwater is not directed to the drainage channel due to the aforementioned issues, these data call for a redesign of the Coker Arboretum drainage channel in order to mitigate flooding problems and comply with the Town of Chapel Hill Design Manual.

CHAPTER 3: SOLUTION IDENTIFICATION AND ANALYSIS

Introduction

This chapter identifies potential solutions to mitigate flooding of the Coker Arboretum drainage channel during a 10 year storm event. Solutions were developed and analyzed through hydraulic and hydrologic modeling, stakeholder consultations, and field measurements. Five technical options are analyzed for flood mitigation during a 10 year storm, with efforts focused on two main approaches, peak flow attenuation and an increase in channel discharge capacity. The set of options is as follows: (1) adjust channel geometry such that discharge capacity is adequate for peak flows; (2) install a detention basin with a gravity outlet at an upstream location; (3) install a detention basin with a pumped outlet at an upstream location; (4) install a detention basin with a gravity outlet at a downstream location; (5) install a detention basin with a pumped outlet at a downstream location. These options are then compared to one another in sufficient detail such that the preferred course of action may be proposed, taking into account flood mitigation effectiveness, environmental impact, stakeholder acceptance, cost and ease of implementation and maintenance.

Hydraulic and Hydrologic Modeling

Hydraulic and hydrologic models were analyzed using Computational Hydraulics International (CHI) PCSWMM, a proprietary user interface for the United States Environmental Protection Agency's (US EPA) Storm Water Management Model (SWMM), which is a widely used industry standard. PCSWMM calculates flow characteristics with the Green-Ampt infiltration method and dynamic wave routing. The dynamic wave routine involves formulating solutions for the gradually-varied unsteady flow equations, also known as the Saint-Venant equations. The unsteady flow continuity equation and the momentum equation are combined and solved along each conduit for each time step. Numerical integration of the two equations is achieved by the Modified Euler Method, allowing for the formulation of solutions that satisfy both equations simultaneously (James, W.; Rossman, L; and James, W. R.; 2010).

Calculation of overland flow routing is accomplished by first determining the typical amount of depression storage depending on subcatchment cover type, imperviousness, and subcatchment slope. Then, once available depression storage has been filled, overland flow is calculated by simultaneously solving the continuity equation and Manning equations, using catchment shape, slope, and roughness as input parameters. Subcatchment time of concentration is then calculated using the kinematic wave formulation, as previously discussed, and used in the peak flow analysis (James, W.; Rossman, L; and James, W. R.; 2010). Relevant tables and figures pertaining to available depression storage and Manning's n-values for overland flow can be found in Appendix C. The Manning equation is also used for open channel flow analysis while the Hazen-Williams equation is used for force main flow analysis.

Stormwater models were adapted for the purposes of this report from a model produced by RK&K Consulting Engineers on a contractual basis with UNC Chapel Hill (RK&K, 2013). The following paragraph describes the input parameters formulated by RK&K Consulting Engineers. Infiltration parameters such as hydraulic conductivity, suction head, and porosity correspond to the soil type characteristics of each respective subcatchment; however, these parameters are not likely to have a significant effect on the overall model unless a timeframe greater than 24 hours were analyzed. Percent slope was calculated using topographical contour lines and a digital terrain model (DTM) based on aerial surveys, while subcatchment imperviousness parameters were based on a GIS layer depicting UNC Chapel Hill land use (RK&K, 2013). Finally, Manning's n values for the drainage channel were assumed to be 0.035 based on the American Society of Civil Engineers (ASCE) Manual of Practice (1982), see Table 4 below, also cited by the SWMM User's Manual (2010). Channel sections are generally concrete or rock bottomed with stone or vegetated banks.

Sensitivity analyses were conducted for all Manning's n values, including channel flow and overland flow for pervious and impervious surfaces. Manning's n for pervious surfaces had the greatest effect; values ranging from 0.15 – 0.4 (corresponding to short, relatively sparse grass and light underbrush respectively) were analyzed. The resulting change in peak flow ranged from an increase of approximately 7% to a decrease of approximately 9%. Interestingly, Manning's n for channel flow had the least effect on peak flow rates, with values ranging from 0.02 – 0.045 (concrete lined to vegetative or natural channels respectively) altering peak flow rates by less than 1%.

Channel Type	Manning n
Lined Channels	
-Asphalt	0.013 – 0.017
-Brick	0.012 – 0.018
-Concrete	0.011 – 0.020
-Rubble or riprap	0.020 – 0.035
-Vegetal	0.030 – 0.040
Excavated or dredged	
-Earth, straight and uniform	0.020 – 0.030
-Earth, winding, fairly uniform	0.025 – 0.040
-Rock	0.030 – 0.045
-Unmaintained	0.050 – 0.045
Natural channels (minor streams, top width at flood stage < 100 ft)	
-Fairly regular section	0.030 – 0.070
-Irregular section with pools	0.040 – 0.100

Table 4: Manning's n Values for Open Channels Based on Channel Characteristics (ASCE, 1982)

Design storms (24 hr. duration) were modeled using values from the NOAA Precipitation Frequency Data Server for weather station Chapel Hill 2 W located at the OWASA facility on Jones Ferry Road, approximately 1.8 miles WSW of the arboretum (NOAA, 2014). The United States Soil Conservation Service (SCS) Type II synthetic rainfall distribution was used to describe the design storms, as dictated by the Town of Chapel Hill Design Manual (2004). SCS rainfall distributions were formulated using historical rainfall data to describe typical storms in various regions of the US. The SCS Type II is one of four synthetic rainfall distributions created to describe four different geographic regions in the U.S., and is the distribution often used to create design storms in the piedmont region of North Carolina. Of the four distributions, SCS Type II features the greatest maximum rainfall intensities for a given 24 hour storm (USDA, 1986). For comparison, Figures 11 and 12 show hourly hyetographs for the SCS Type II 10 year 24 hour design storm and actual rainfall data from June 30, 2013 taken at the previously described KIGX weather station. Figure 11 displays the data in terms of a cumulative rainfall percentage throughout the duration of the storm, while Figure 12 shows hourly precipitation volumes.

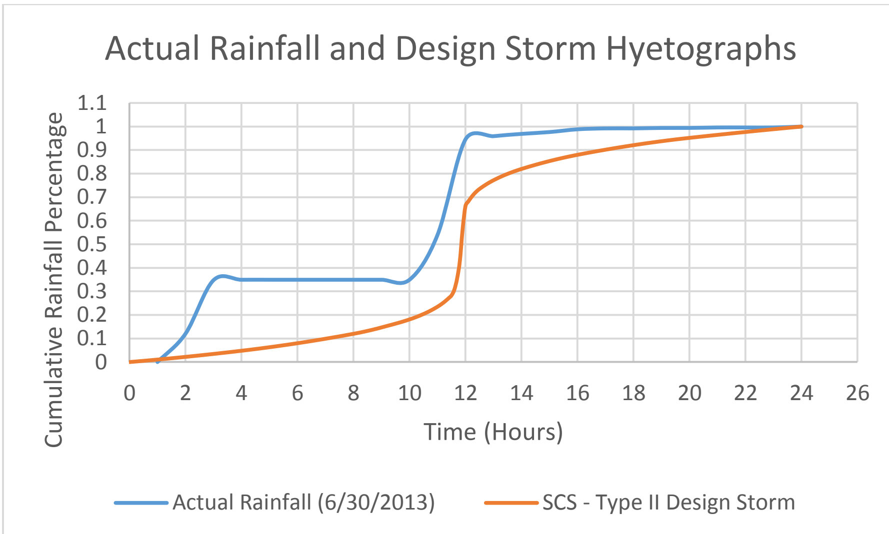

Figure 11: Cumulative Rainfall Percentage Hyetographs of 6/30/2013 Precipitation Data and the SCS Type II 10 yr 24 hr Design Storm

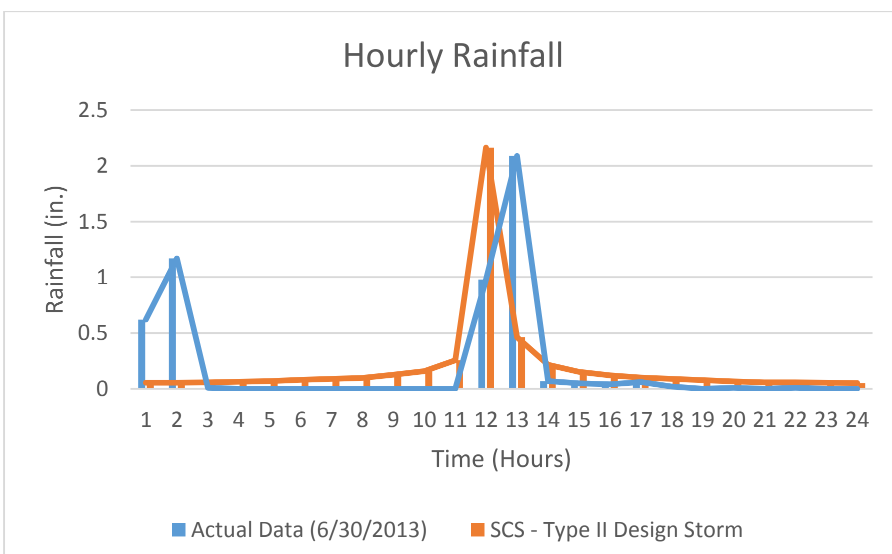

Figure 12: Hourly Rainfall Hyetographs of 6/30/2013 Precipitation Data and the SCS Type II 10 yr 24 hr Design Storm

In order to more completely tie design work into the specific subcatchment parameters, the 10 year 24 hour SCS – Type II design storm was compared to NOAA precipitation frequency estimates for durations corresponding to the subcatchment times of concentration. Once the time of concentration is reached, flow rates level off and reach an equilibrium, so in order to truly be considered a 10 year storm in terms of the subcatchments, frequency estimates must be determined for a storm duration equal to the subcatchment times of concentration. Table 5 shows that the most intense durations of the 24 hour SCS – Type II design storm are comparable to the NOAA 10 year estimates, and are consistently higher with a percent difference of up to 11%.

Duration (min)	Max. SCS Intensity (24 hr Duration) (in/hr)	NOAA 10 yr Estimate Intensity (in/hr)	% Difference
60	2.35	2.35	0%
30	3.93	3.68	7%
18	5.32	4.78	11%
12	6.00	5.55	8%
6	7.09	6.84	4%

Table 5: Comparison of the Most Intense Durations within the 24 hr. SCS - Type II Design Storm that Correspond to the Subcatchment Times of Concentration and the NOAA 10 yr. Intensity Estimates for the Same Durations

Design storm return intervals and corresponding 24 hour rainfall volumes are as follows in Table 6, along with model output peak flow rates for the channel section that experiences flooding during large storm events, with the same information represented as a curve in Figure 13.

Return Interval (yr)	24 hr Rainfall Volume (in)	Peak Flow in Flooded Channel Section (cfs)
1	2.96	19.8
10	5.17	32.0
25	6.11	44.7
50	6.86	49.0
100	7.62	51.8

Table 6: Peak Flow Rates Associated with 24 hr. Duration Design Storms

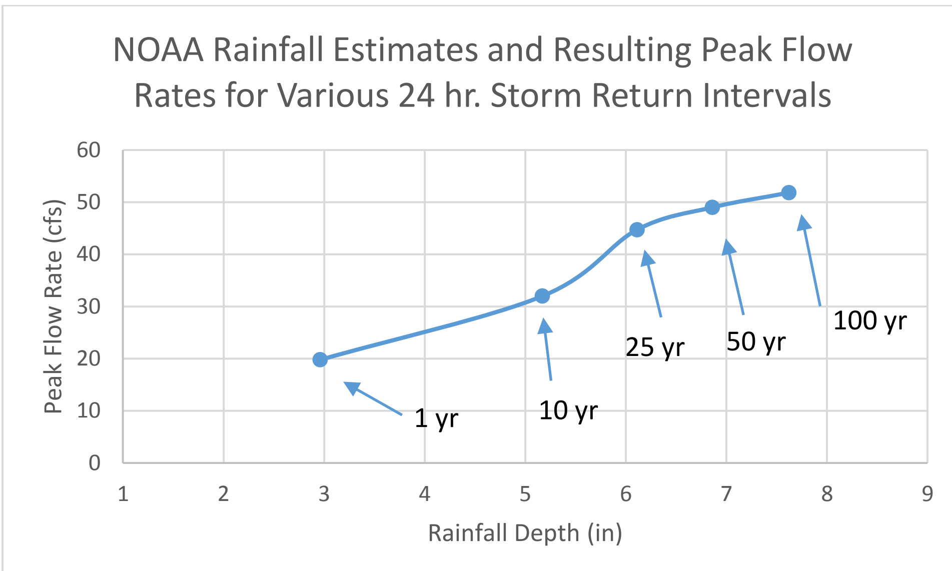

Figure 13: Curve of NOAA Rainfall Estimates vs Peak Channel Flow Rates in the Channel Section of Concern

Option Design and Analysis

Increasing Discharge Capacity

Channel redesign parameters were selected to maintain upstream and downstream cross-sectional uniformity and to minimize impact in terms of required grading, excavation, and backfilling. Proposed channel geometry is shown in Figure 14, while steady state discharge capacity and 10 year peak flows can be seen in Table 7, both with and without adherence to the freeboard criterion. A freeboard of 0.3 ft. was included in the channel redesign, as stipulated by the Erosion & Sediment Control/Stormwater Certification workshop created by the Biological & Agricultural Engineering and Soil Science Departments at North Carolina State University (NCSU) in partnership with the North Carolina Department of Transportation (NCDOT) (2006). This freeboard criterion was developed by Glenn Schwab and his colleagues in the technical reference text entitled *Soil and Water Conservation Engineering* (1966) and is also used by the Purdue Engineering Department in the web-based publication “Technical Information for a Concrete Lined Channel” (n.d.). In order to attain the parameters shown in Figure 14, either an entire tree or at least some root material must be removed at cross-section #4. The rest of the channel will require only excavation and grading.

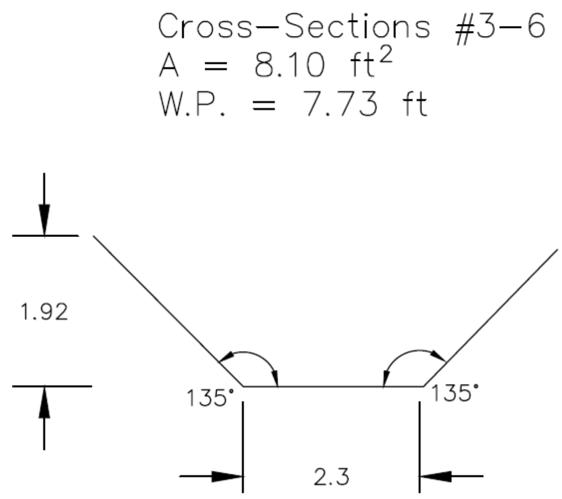

Figure 14: Proposed Channel Geometry

Cross-Section	Discharge Capacity (cfs) Without Freeboard	Discharge Capcity (cfs) With Freeboard	10 year Peak Flow (cfs)
#3 – 6	45.0	32.0	32.0

Table 7: Discharge Capacity of Proposed Channel, with and without freeboard, Compared to Peak Flow Rate

With these parameters, the channel meets the design criterion of the 10 year design storm laid out by the Chapel Hill Design Manual, NCSU, NCDOT, and Schwab, et al, with the just over 0.3 ft. of freeboard creating an excess capacity of about 40%. Under these conditions, when the channel is filled to the tops of the banks there is sufficient capacity to handle the peak flow rate of the 25 year storm event. Furthermore, the proposed discharge capacity is within about 8% and 13% of the peak flow rates for the 50 year and 100 year storm events, respectively. Therefore, designing the channel for the 100 year storm would likely produce only a small amount of additional costs, labor, and environmental impact, but the downstream network is unlikely to be designed for the 100 year storm so flooding issues would effectively be shifted downstream.

Peak Flow Attenuation

Detention basins are a commonly used flood mitigation measure. They combine storage with the regulated release of water in order to limit downstream peak flows. Installation of a detention basin is proposed at one of two possible locations. The first location (basin #1) shown in Figure 15 is towards the upstream end of the arboretum, near the beginning of the drainage channel. The proposed location of the second option (basin #2) shown in Figure 16 is near the center of the arboretum. These two locations were chosen due to their relative lack of geographic constraints and their effect on the channel sections known to experience flooding during major storm events. The basin cannot be installed any farther upstream without daylighting a length of conduit and greatly increasing impact and costs, and if moved farther downstream, the basin would have no effect on the problematic channel section. Each basin was analyzed for its hydraulic effect both with and without an installed pump.

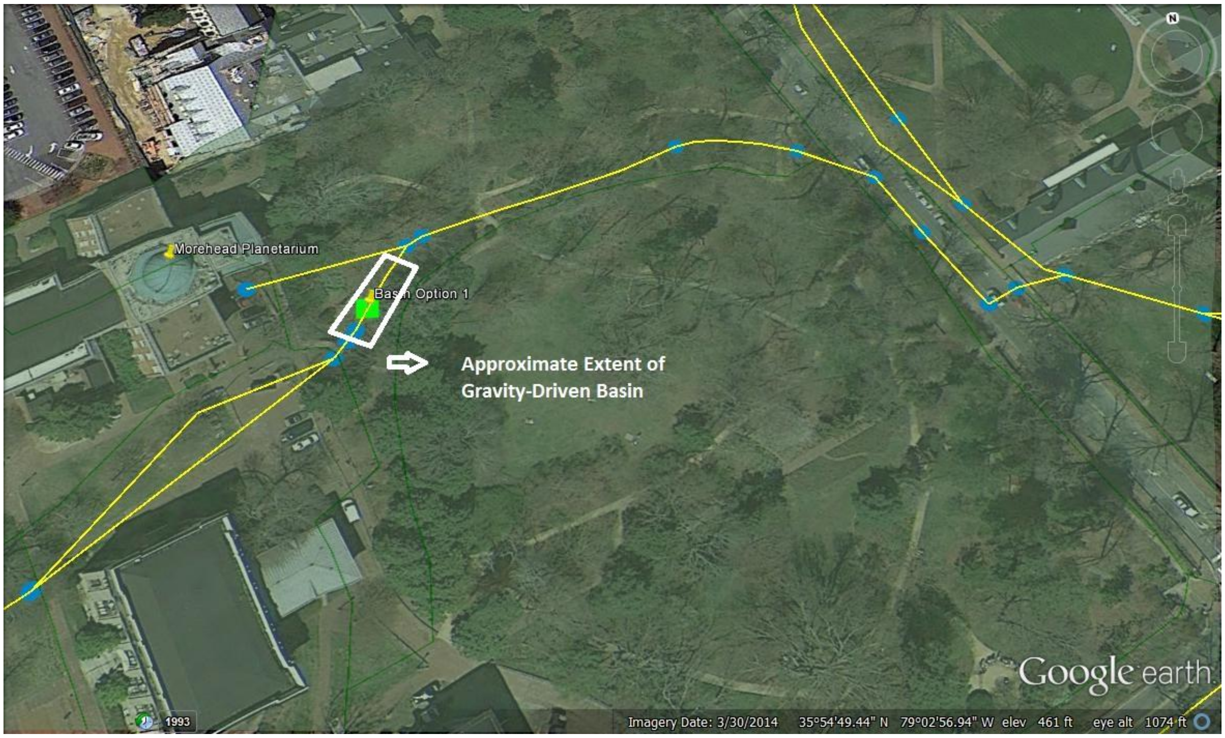

Figure 15: Location Option #1 for Proposed Detention Basin Option

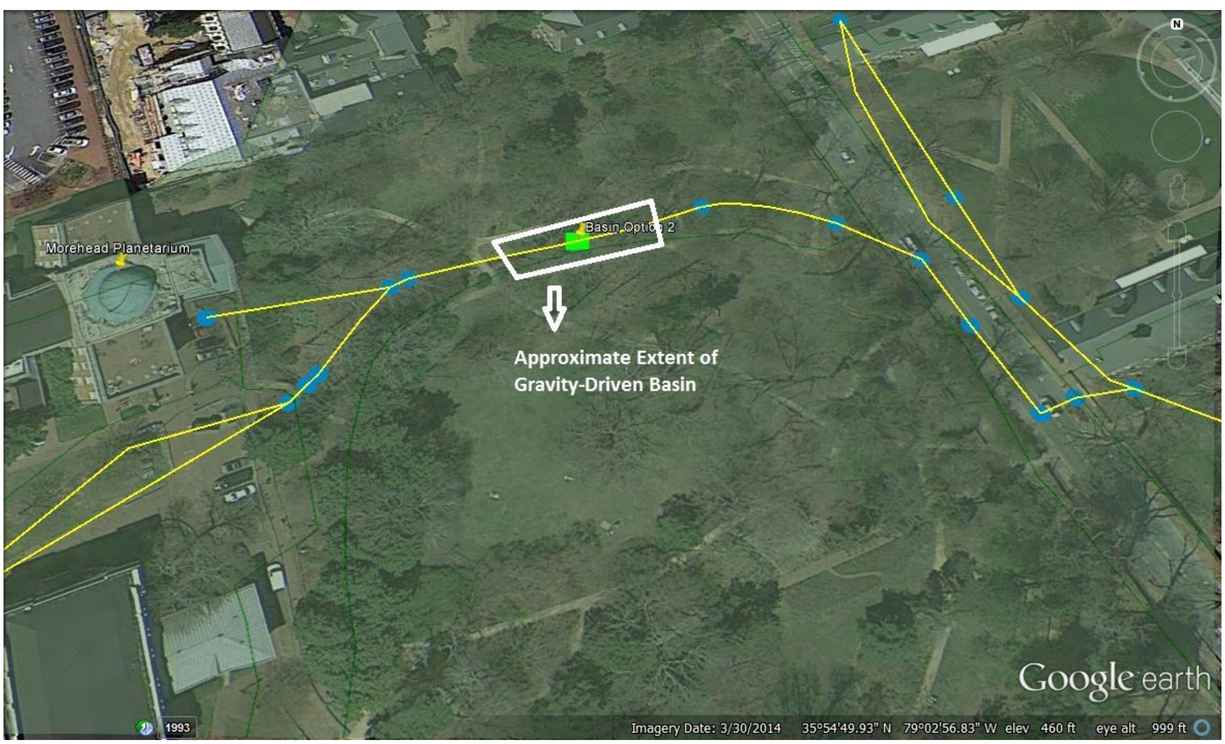

Figure 16: Location Option #2 for Proposed Detention Basin Option

Detention Basin with Gravity Outlet

Detention basins with gravity outlets do not rely on a pumping system to control the release of downstream flows. Instead, stormwater enters at the upstream end, and as the basin fills, the stormwater is drained by gravity through a submerged outlet located at the bottom of the downstream end. The outlet is designed to release a specific maximum outflow from the basin. For the Coker Arboretum gravity-driven detention basin, a length of channel would be widened and graded to the elevation of the downstream end so that basin invert and sidewall elevations are fixed throughout the length. Lastly, a dam would be placed at the downstream end to create storage capacity, with an outlet passing through the bottom of the dam to limit downstream flows rates, as shown in Figure 17 below.

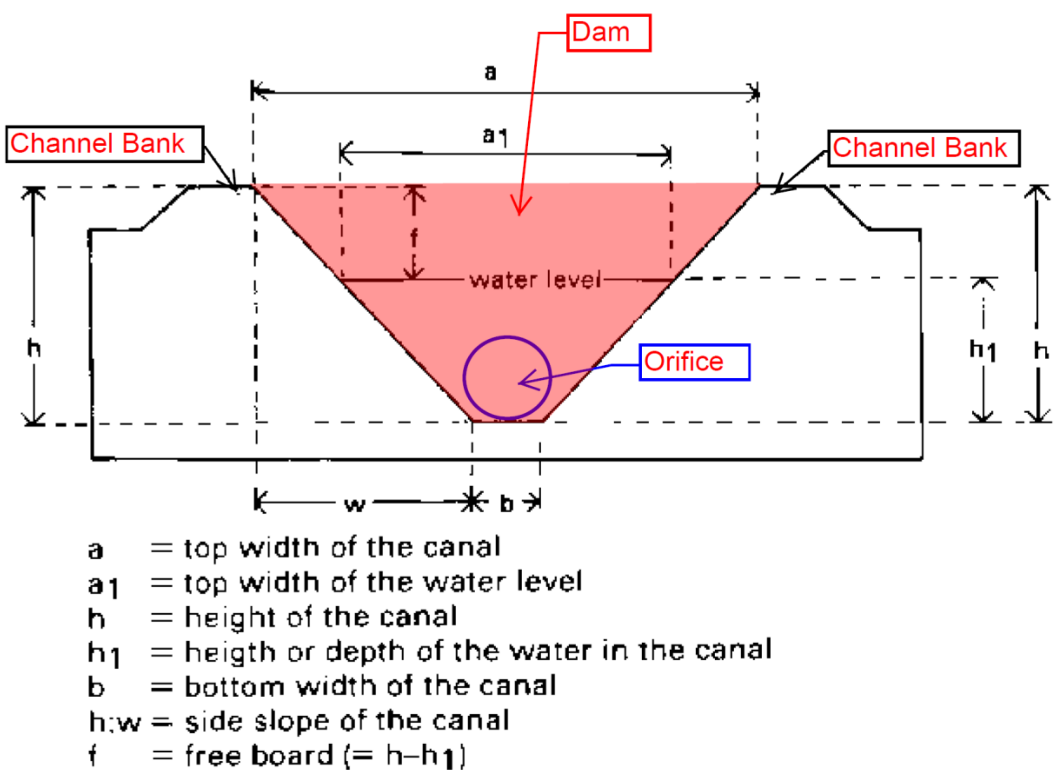

Figure 17: Typical Cross-Section of a Gravity Dam Outlet (FAO, 1985)

The PCSWMM storage pond calculator was used to conduct a storage balance on basin volume during a 10 year storm event in order to determine the approximate storage necessary for flood mitigation at various design outflows. Next, channel slope and length were used to calculate the maximum allowable depth of the detention basin. Basins were modeled to feature various outlet diameters, and the orifice equation was used to calculate outlet flow as a function of basin depth, in accordance with the City of Raleigh Stormwater Design Manual (2002). Finally, basin volumes attained from the storage pond calculator tool were adjusted to minimize required storage volume while maintaining flood mitigation and 0.3 ft. of freeboard. Due to already limited basin capacity, a permanent pool was not included in the design. Model results can be seen in Table 8 below, while Table 9 shows basin parameters. A 1:1 side slope ratio was chosen to maintain consistency with the rest of the channel. Optimal outlet diameters were selected to release stormwater at the highest rate possible while limiting the need for downstream channel alterations.

Basin Location	Outlet Diameter (in.)	Storage Required (ft^3)	Peak Outlet Flow (cfs)	Limiting Discharge Capacity (cfs)
Basin #1	10	36,400	10.8	8.07
	12	33,152	12.0	8.07
	14	30,128	14.2	8.07
Basin #2	16	33,022	11.3	18.2
	18	28,633	14.1	18.2
	20	24,244	16.6	18.2

Table 8: Gravity Basin Model Results Compared to Existing Discharge Capacities

Basin Location	Outlet Diameter (in.)	Allowable Channel Length (ft)	Allowable Depth (ft)	Base Width (ft)	Side Slope (run:rise)	Top Width (ft)	x-sec. Area (ft^2)	Volume (ft^3)
Basin #1	12	85.5	2.24	171	1	**175**	388	33,152
Basin #2	20	130	2.09	87.1	1	**91.3**	186	24,244

Table 9: Gravity Basin Parameters Necessary to Achieve Certain Required Storages

As shown in Table 9, the maximum length and depth dimensions of the basin are fixed due to geographical and topographical constraints. Therefore, the only design parameter available to meet the computed required storage volumes is the width. If a detention basin with a gravity outlet were installed, the impact on the arboretum would be substantial, with a minimum width of over 90 feet. Basin location #1 is bordered on either side by walking trails, limiting the allowable width to approximately 30 ft. Basin #2 is somewhat less constrained, but should be limited to 40 ft. in order to minimize the need to remove large trees and otherwise disturb landscape installations. Furthermore, even if basin geometry did not exceed maximum allowable widths, alterations to channel geometry would still be necessary at various cross-section locations if a detention basin with a gravity outlet were installed at location #1. A gravity-driven detention basin alone cannot achieve acceptable levels of peak flow attenuation.

Detention Basin with Pumped Outlet

Installment of a pump-driven detention basin would allow for greater basin depth and thus a smaller footprint. Pumps were modeled after Xylem brand column pumps (2015). This type of pump can produce high flows at low head, is often used for flood control and can feature a water level sensor for automated startup and shutoff. Ten pump models were assessed along with two different pipe diameters to create the system head curves shown in Figures 18 and 19. Pumps are indicated in the legend by their model numbers and each system curve represents the head required to pump water to the top of the basin as water level rises and static lift is reduced, while also accounting for friction and minor losses. From the system head curves, pump operating points were determined and used to model pump curves in PCSWMM as a function of basin depth. Pumping systems were then modeled at each potential location as a pump and a weir to account for emergency basin overflow.

As with the gravity systems, the PCSWMM storage pond calculator tool was used to determine preliminary storage requirement values and then models were run to determine more accurate volume requirements. Maximum basin depth was set at eight feet to limit environmental impact and, because pumped outlets allow for larger basins, side slopes were limited to a 2:1 run:rise ratio for stability reasons. The pump intake was modeled one foot above the basin floor to reduce clogging and other maintenance issues, and a freeboard of one foot was included in the design criteria.

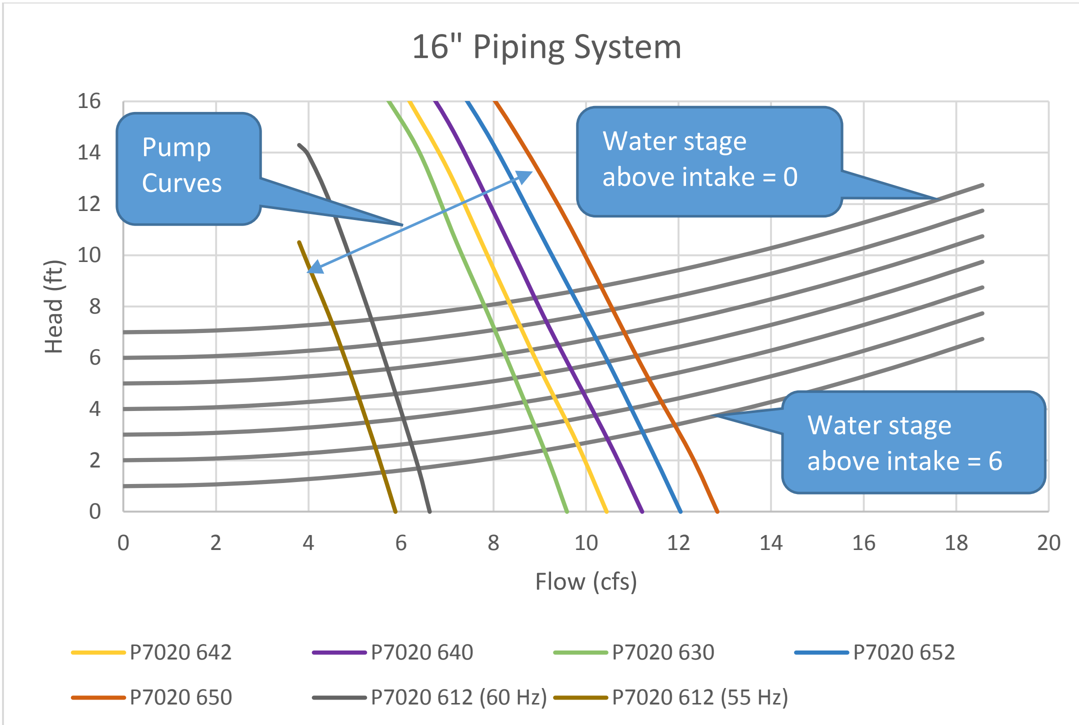

Figure 18: System Head Curves for 7 Pump Models and a 16" Piping System

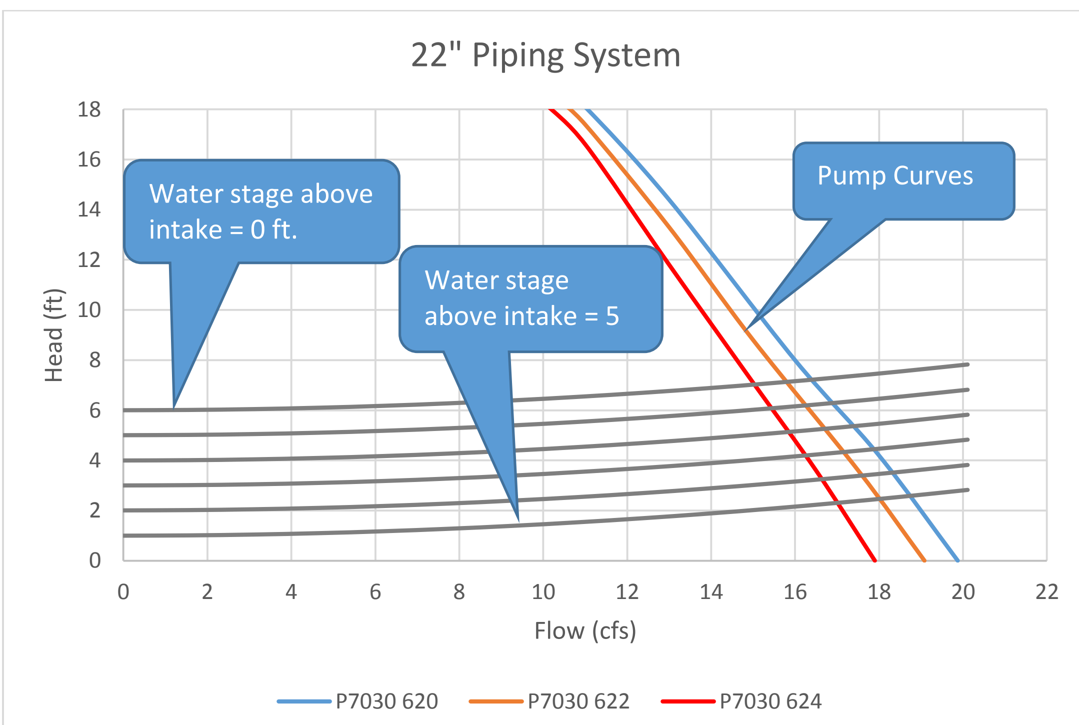

Figure 19: System Head Curves for 3 Pump Models and a 22" Piping System

Table 10 shows the storage volume required for various pumps as well as the resulting peak downstream flows and existing discharge capacities, while Table 11 shows potential basin parameters to meet storage requirements during a 10 year storm. Optimal pumping systems were selected to release stormwater at the highest possible rate while limiting the need for downstream channel alterations. Note that Basin #2 has a maximum depth of 7 feet due to width constraints.

Basin Location	Pump Model	Basin Storage Required (ft^3)	Peak Outflow (cfs)	Limiting Discharge Capacity (cfs)
Basin #1	P7020 612 (55 Hz)	25,165	13.7	8.07
	P7020 612 (60 Hz)	23,345	14.6	8.07
Basin #2	P 7030 620	6,126	18.9	18.2
	P 7030 622	7,032	18.0	18.2
	P 7030 624	8,292	17.0	18.2

Table 10: Pumped Basin Model Results Compared to Existing Discharge Capacities

Basin Location	Pump Model	Channel Length (ft)	Basin Depth (ft)	Base Width (ft)	Side Slope (run:rise)	Top Width (ft)	x-sec. Area (ft^2)	Volume (ft^3) w/ 1 ft freeboard
Basin #1	P7020 612 (55 Hz)	80	8	8	2	**40**	192	12,320
Basin #2	P 7030 622	85	7	2	2	**30**	112	7,140
		110	6	3	2	**27**	90	7,150

Table 11: Basin Parameters Necessary to Achieve Certain Required Storages

As seen in Tables 10 and 11, Basin #1 can only provide around half of the required detention storage and significant downstream channel alterations would still be necessary. Basin #1 with a pumped outlet is not a viable option for flood mitigation in the Coker Arboretum drainage channel. Basin #2 works functionally because it is possible to release stormwater at a much higher rate without having to increase downstream channel discharge capacity. However, Basin #2 with a pumped outlet raises other concerns in terms of environmental impact, public safety, stakeholder acceptance and capital and maintenance costs.

The basin, under the proposed parameters described in Table 11, would be at least six feet deep and, at a minimum of 27 feet wide, would be bordered closely on either side by walking trails. This would greatly increase environmental impact and excavation costs compared to the channel redesign option and could raise issues of public safety for arboretum visitors. A portion of the natural areas that arboretum staff have worked to cultivate would need to be permanently removed to make room for the basin. Also, for reasons of liability and public safety it would be prudent to include a fence around the basin that would closely border two walking trails and potentially cause further disruption to the natural environment that visitors enjoy. Furthermore, if Basin #2 were installed a second pump for standby capacity in the event of maintenance and repair issues should be considered. Installation of two pumps with the necessary capacities described above would significantly increase capital and O&M costs.

CHAPTER 4: SOLUTION COMPARISON

The relative merits of each flood control strategy are shown in Table 12 below, along with a qualitative score. Scoring criteria are color-coded above the table, with all criteria weighted equally with the exception of flood control effectiveness because this is ultimately the most important criterion.

	Flood Control	Environmental Impact	Relative Cost and Ease of Implementation	Stakeholder Acceptance	Cost and Difficulty of O&M	
	0 pts	0 pts				
	5 pts	3 pts				
	10 pts	5 pts				
Options	**Flood Control**	**Environmental Impact**	**Relative Cost and Ease of Implementation**	**Stakeholder Acceptance**	**Cost and Difficulty of O&M**	**Qualitative Score**
Basin #1, Gravity Outlet	Low	High	Medium	Medium	Low	**11**
Basin #2, Gravity Outlet	Low	High	Medium	Medium	Low	**11**
Basin #1, Pumped Outlet	Low	Medium	High	Low	Medium	**6**
Basin #2, Pumped Outlet	High	Medium	High	Low	Medium	**16**
Increase Channel Discharge Capacity	High	Medium	Medium	High	Low	**26**

Table 12: Flood Control Strategy Relative Comparison Criteria

Installation of a detention basin with a gravity outlet at either location cannot achieve acceptable levels of peak flow attenuation to reduce flooding in the Coker Arboretum under the geographic constraints described in Chapter 3. Neither can installation of a basin with a pumped outlet at the upstream location due to the limited rate at which stormwater can be released from the basin to minimize the need for downstream channel alterations. Thus, there is no need for further comparison of these three options. The following chapter will further compare the options of: (1) an increase in discharge capacity and (2) installation of a detention basin and pumping infrastructure at the downstream location option.

Basin #2 with a pumped outlet would work functionally, but raises a number of issues, most importantly increased capital and O&M costs but also concerns regarding environmental impact, public safety, and stakeholder acceptance. A planning level cost estimate for the conceptual designs presented in Chapter 3 revealed that purchase and installation of detention basin pump infrastructure would cost around $40,000 – 50,000 according to Dan Joyce, the sales engineer for this region of North Carolina. If a second pump were installed in order to provide backup capacity in the event of pump failure or maintenance downtime, the cost would likely approach $100,000 for pump infrastructure alone, not to mention operation and maintenance costs. Furthermore, increasing channel discharge capacity would require approximately 10 cubic yards of excavation, while about 264 cubic yards of excavation would be necessary to attain the proper amount of storage volume for the detention basin. It is estimated that the more than 26-fold increase in excavation is would raise capital costs an additional $6,000. Many other costing parameters would remain relatively comparable for the two projects.

Due to the considerations described above, an increase in discharge capacity should be explored in more detail in order to comply with the Chapel Hill Design Manual and thus mitigate flooding in the Coker Arboretum during a 10 year SCS – Type II design storm with a 24 hour duration. In this way, it is possible that flood issues may be alleviated in the least costly and safest manner while continuing to maintain and protect the landscaped environment of the Coker Arboretum and Botanical Garden.

CHAPTER 5: IMPLEMENTATION OVERVIEW

Introduction

Chapters 3 and 4 determined that the specifics involved in increasing drainage channel capacity should be examined in greater detail. This chapter will discuss factors relevant to implementing the project, including consideration of review and permitting processes, construction, scheduling, project area disruptions, resource requirements, and total costs. The majority of the information in this chapter came from a September 22nd, 2015 meeting with UNC stormwater engineer Sally Hoyt as well as an October 9th, 2015 meeting Margo Macintyre and Geoffrey Neal, the curator and assistant curator of the Coker Arboretum, respectively.

Review and Permitting

This is considered a relatively small project by the UNC Energy Services Department (ESD) and is likely to fall well within the department's budget for maintenance, repairs, and project implementation. As such, only an internal review will be necessary, with no required administrative review at the municipal, county, or state level. The project will most likely be reviewed by Sally Hoyt, a stormwater engineer with the UNC ESD, the curator of the Coker Arboretum, Margo MacIntyre, and the UNC Environmental Health and Safety Department (EHS). Additionally, if the projected is selected to move forward, further design will take place in order to review and finalize the conceptual designs presented in the Chapter 3.

This additional design work will most likely not be conducted by ESD, but rather by one of a number of civil engineering firms that are engaged in an open-ended design contract with the university. The firms involved in this contract were selected through a competitive process, so no request for proposals will be necessary. The firm that is selected for and agrees to implement the project will be responsible for, other than the additional design work, the production of construction documents and the carrying out of construction management. Furthermore, a landscape architect may be consulted to review the post-construction planting plan.

Construction and/or maintenance of any kind that occurs in or around waterways of any type are subject to compliance with nationwide permits (NWPs) in coordination with the North Carolina Department of Environmental Quality (NCDEQ), formerly the North Carolina Department of Environment and Natural Resources, and the Army Corps of Engineers (Corps). Over 50 NWPs exist, and the necessary compliance depends on the type of project and the conditions under which it is undertaken. Due to the nature, size, and scope of the proposed project, it will most likely require only NWP 3 – Maintenance. NWP 3 pertains to “The repair, rehabilitation, or replacement of any previously authorized, currently serviceable structure…” and allows for “Minor deviations in the structure’s configuration or filled area…” (Army Corps of Engineers, 2012). The project will not affect any jurisdictional wetlands or cause further loss of any perennial, intermittent, or ephemeral stream bed. Under these conditions, no further wetland and waterway “Waters of the United States” permitting compliance or preconstruction notifications are necessary. Although NWP 3 is most likely the only permit that will be required, the Corps and NCDEQ should be consulted to ensure the correct compliance. EHS will be responsible for coordinating any required nationwide permitting conditions.

As shown by the Federal Emergency Management Agency (FEMA) floodplain map (Figure 20 below), the project area is not within a 100-year floodplain, which are shown in light blue in Figure 20, so no Federal floodplain management requirements are applicable (FEMA, 2015). The closest special flood hazard area (100 year floodplain) is in the floodplain of Battle Branch, a significant distance to the east of the arboretum. Lastly, an erosion control permit will not be required because the area of impact will be less than one acre. However, an erosion control plan must be produced by the supervising engineer and approved by EHS.

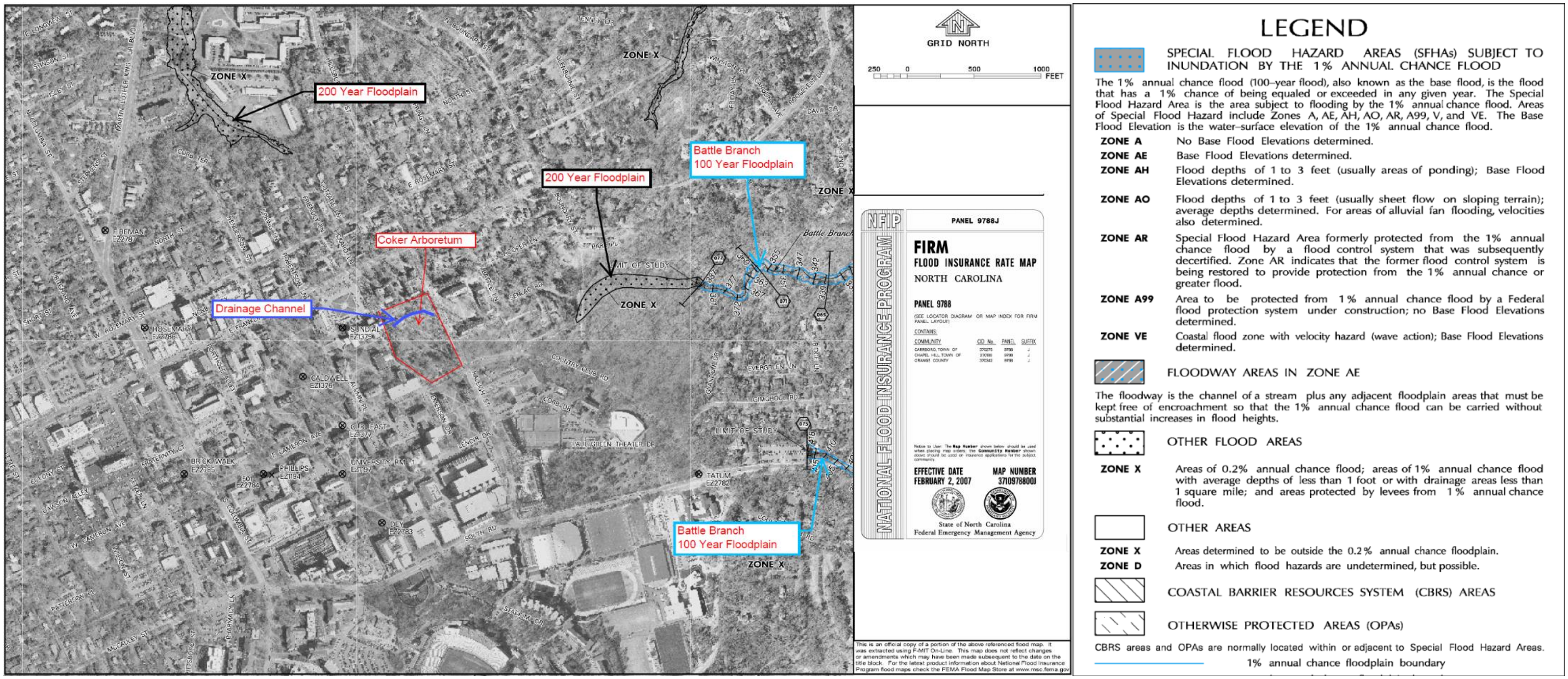

Figure 20: FEMA Flood Rate Insurance Map

Construction

The general construction process will occur as follows. The site will be prepared by removing any vegetation that arboretum staff decides should not be damaged. Such vegetation will be safely stored until it is replanted post-construction. Then a mini excavator will be used to widen and otherwise excavate the channel according to the design sections shown in Chapter 3. Excavated material will be hauled offsite and sediment runoff will be mitigated with silt fencing in sensitive areas such as stockpiles and walkways.

During construction in perennial or intermittent streambeds, a pumping system is often needed to transport base flow around the construction area. Flow is halted with sandbags and an intake is located upstream of the project area. Temporary piping then transports water around the site to reenter the channel downstream of the project area. This produces the dry conditions needed for channel construction. However, because flow through the section of the Coker arboretum channel that will experience construction is ephemeral, no pump around system will be required unless the sump pump in the basement of the Morehead Planetarium is active, which feeds directly into the upstream end of the channel and effectively creates base flow conditions.

By the end of each work day, excavated banks will be stabilized with a biodegradable coconut fiber matting and planted with a temporary riparian seed mix to mitigate the risk of future erosion (CWP, 2004; Hoyt, 2015). The seed mix should contain native species to encourage vegetation establishment in the riparian area while limiting the possibility that invasive species will be introduced (CWP, 2004; MacIntyre, 2015). Upon completion of excavation and stabilization, any demolished stonework will be replaced, then the banks of the channel will be replanted with more permanent vegetation either from the previously removed transplants or new plantings depending on the area in question.

According to Margo MacIntyre, the curator of the Coker Arboretum, the arboretum staff will be responsible for all pre- and post-construction activities concerning the affected landscaped areas around the site. They will remove and store the necessary vegetation in order to provide equipment access to the channel, then either replant the transplants or provide new plantings, depending on the area in question. This is necessary to maintain the specific landscapes that the arboretum and its staff strive to cultivate. The funds for this aspect of the project will likely be supplied by the arboretum's normal operating budget.

Due to the sensitive and specialized nature of waterway construction, the project will most likely not be implemented by UNC construction shops, but rather by qualified and experienced contractors. However, with about 150 feet of channel affected, the limited size and scope of the project makes a prequalification process unnecessary. Because earthwork requirements are small, with about 10 cubic yards of excavation and 130 square yards of grading, a subsurface utility survey will not be needed, but utility location services should be carried out to ensure that no utilities will be affected by project implementation. Utility location services are provided by utility companies at no cost as a required component of the contractor's preconstruction due diligence.

The staging area for construction will be along the access road that borders the arboretum to the west, most likely behind Howell or Davie Hall, where other construction staging activities have occurred in the past. This will provide easy access to the site through one of the western entrances located near the upstream end of the project while avoiding issues with vehicle traffic. The entrance directly behind Morehead Planetarium is closest to the site, but is bordered by stone pillars that would make access with a mini excavator difficult. Furthermore, the first portion of the path at this entrance (which was recently renovated with stone pavers) may become damaged with regular equipment traffic. The next entrance to the south has, according to Margo MacIntyre, been used as a small equipment access point in the past and is better because it is more spacious and features gravel construction. This entrance is also closer to the potential staging areas, reducing traffic disruption on the access road to the south of Morehead Planetarium during times of equipment and material mobilization.

Scheduling

The overall timeline and schedule of the project will ultimately be decided by the construction contractor and the supervising engineer along with ESD and arboretum staff. Construction of the project will likely take one to three weeks depending on weather conditions, unforeseen excavation issues such as large rocks and boulders, onsite accidents, and equipment downtime due to unforeseen repairs. The ideal time of the year for the project to be implemented is in the winter for a number of reasons.

Most importantly, there is a reduced chance of heavy storm events in the winter, which could disrupt the construction process by way of undesirable working conditions, flooding, and limitation of equipment access, as well as produce increase risks of bank erosion and other sources of sediment runoff. Transplanting and construction will also have less impact on affected vegetation during the winter months. During this time, vegetation will be more or less dormant and therefore less likely to be damaged when transplanted, trampled, or otherwise impacted by construction activities. Additionally, the arboretum receives the least amount of visitors in the winter months, so public disruption will be kept to a minimum. Public disruption would be reduced even further if the project were implemented over winter break, when the access road to the west of the site behind Davie Hall, Howell Hall, and the Morehead Planetarium is experiencing minimal traffic.

Public Disruption

The site is directly bordered to the north and south by walking trails within the arboretum, both of which would be closed along the extent of the site for the duration of construction. Furthermore, the corridor between Howell and/or Davie Halls and the construction entrance to the arboretum may be briefly impacted when materials and equipment are being mobilized from the staging area to the construction site. A pedestrian detour plan will need to be implemented by the contractor, consisting mainly of detour signs on the walking paths and possibly some blaze orange safety fencing. It will be the responsibility of the UNC Department of Transportation and Parking (T&P) to notify the relevant parties affected by the placement of the staging area and it may be necessary to pay UNC T&P if any parking spaces are affected, according to how many spaces are affected and for how long.

Resource Requirements

The resources required for the proposed project include silt and safety fencing, inlet protection, coconut fiber matting, temporary riparian seed mix, mortar, and field stone. As previously explained, a pump around system and sand bags will mostly likely not be necessary. In addition to the construction materials described above, the project will require light-duty construction equipment such as a mini excavator, hand tools and labor.

Operation and Maintenance

The proposed solution to mitigate flooding of the Coker Arboretum drainage channel during heavy storm events is not mechanical in nature and will produce no further maintenance burden on arboretum staff. That is not to say that channel maintenance of any kind will not be necessary, but rather that the proposed project will not create the need for any additional maintenance beyond what arboretum staff are already responsible for. Therefore, O&M costs are assumed to be negligible.

Capital Costs

The total cost of channel redesign and construction includes only capital costs. Because the project will require no additional operation and maintenance costs, they are not included in this report. Total capital costs for design and construction are shown in Table 13 below. Unit abbreviations are as follows: Each (EA), Linear Foot (LF), Cubic Yard (CY), Square Yard (SY), Acre (AC), and Lump Sum (LS). Total construction costs are estimated at about $16,500, while overall capital costs including additional design and construction management are estimated at about $26,500. The conceptual level design calculations used to estimate costing parameters are presented in Appendix D.

Construction costs include site preparation, earthwork, sediment and erosion control, bank and bed stabilization, and site management. Site preparation costs include tree removal and stump removal as well as safety fencing to alert pedestrian traffic and protect any vegetation that is not removed. The tree and stump removal refers to an arborist's estimate to remove the problematic gum tree mentioned in Chapters 2 and 3, whose roots have constricted the channel, reducing discharge capacity and therefore causing backwater and exacerbating flooding issues.

Earthwork costs include demolition, excavation, and grading. Demolition refers to the removal of the existing stone and concrete that lines the channel bed. For the purposes of cost calculation, the bed lining material was assumed to be six inches thick on average. Excavation includes the removal of bank material in order to achieve the proposed geometry, and was calculated by taking the difference in area between existing and proposed channel cross-sections, as depicted by the blue hatching in Figure 21, over the length of the channel. After excavation, the channel bed and banks will be graded to ensure that a uniform slope is obtained to reduce bottlenecks.

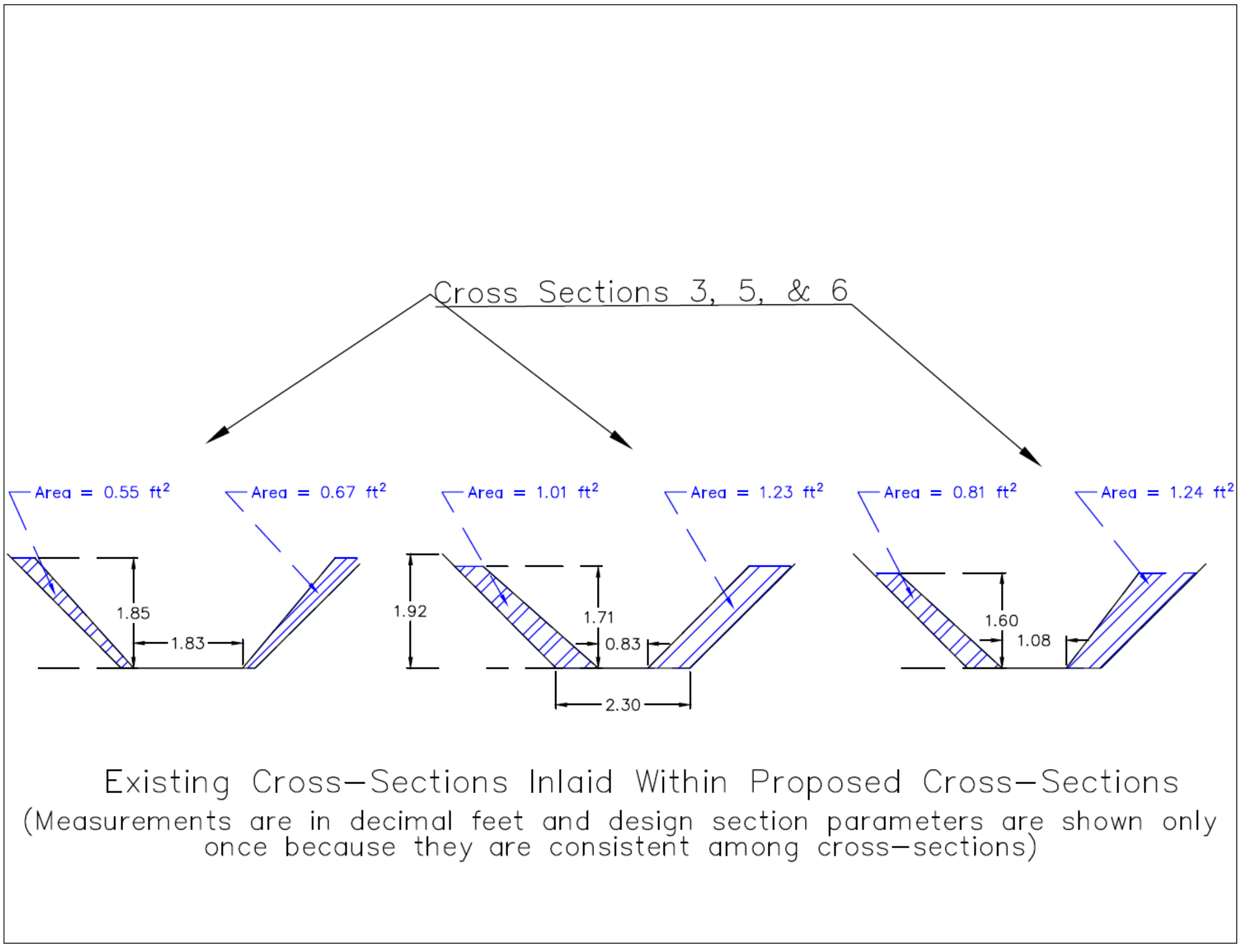

Figure 21: Excavation Costing Parameters, with Existing Channel Cross-Sections Superimposed within Proposed Cross-Section

Sediment and erosion control includes inlet protection for the culvert at the downstream end of the channel, as well as silt fencing to encompass sensitive runoff areas such as walkways and stockpiles of excavated material. Bank and bed stabilization includes installing coir fiber matting and seeding with a temporary riparian seed mix to deter bank erosion and sediment runoff, as well as replacing the stonework that lines the bottom of the channel to stabilize the bed and deter unwanted vegetation. It is proposed that the stonework channel lining be replaced with concrete as a cost saving measure because the required masonry work is estimated to cost over $10,000, based on prior work contracted by the arboretum, while the concrete lining is estimated at about $1,500. Site management includes pedestrian traffic control and refers to implementation of the previously described pedestrian detour plan.

Mobilization and demobilization of equipment and materials is estimated to be about 10% of total construction costs, or about $1,500. Finally, UNC stormwater engineer Sally Hoyt estimates that the additional project design and construction management will cost approximately $10,000. The majority of unit cost data in Table 13, other than tree and stump removal, grading, and concrete lining installation, is based on estimates for comparable projects compiled by either the UNC ESD or Wildlands Engineering of Raleigh, NC. As mentioned earlier, the price of tree and stump removal is based on bids given by contractors to Margo MacIntyre after site visits. Channel grading cost estimates are based on the Center for Watershed Protection's Urban Subwatershed Restoration Manual Series, Manual #4 – Urban Stream Repair Practices (2004). Lastly, the cost to install a concrete lining on a portion of the channel cross-section is based on the Purdue University Department of Engineering web-based publication "Technical Information for a Concrete Lined Channel" (n.d.).

Item	Quantity	Unit	Unit Cost	Total Cost
Site Preparation				
Tree Removal by Arborist	1	EA	$ 6,000.00	$ 6,000.00
Tree Protection/Safety Fence	200	LF	$ 3.00	$ 600.00
Stump Grinding	1	EA	$ 500.00	$ 500.00
Earthwork				
Demolition	8	CY	$ 50.00	$ 400.00
Excavation and Disposal	10	CY	$ 25.00	$ 250.00
Grading	130	SY	$ 15.00	$ 1,950.00
Sediment and Erosion Control				
Silt Fence	150	LF	$ 3.00	$ 450.00
Inlet Protection	1	EA	$ 100.00	$ 100.00
Bed & Bank Stabilization				
Coir Fiber Matting	200	SY	$ 4.00	$ 800.00
Temporary Riparian Seed	0.14	AC	$ 500.00	$ 70.00
Concrete Lining	570	SF	$ 2.50	$ 1,425.00
Site Manaagement				
Pedestrian Traffic Control	1	LS	$ 1,000.00	$ 1,000.00
Parking	1	LS	$ 1,500.00	$ 1,500.00
Subtotal				$ 15,045.00
Mobilization and Demobilization (10% of Subtotal)				$ 1,504.50
			Construction Cost	$ 16,549.50
Additional Design and Construction Management				$ 10,000.00
			Total Cost	$ 26,549.50

Table 13: Cost Estimate of Project Implementation, Based Largely on Project Data from UNC ESD and Wildland Engineering

The total cost of mobilization and demobilization, construction, design, and management is estimated to be about $26,500. ESD has access to a Stormwater Utility budget for utility maintenance and project implementation at the discretion of the department. Funds for the UNC Stormwater Utility budget are collected by billing internal users of the utility and total about $250,000 per year. At around $26,500, the proposed solution to mitigate drainage channel flooding in the Coker Arboretum is well within the means of the Stormwater Utility budget.

Cost Benefit Analysis

The majority of damage caused by drainage channel flooding in the Coker Arboretum is restricted to the washout of walking paths and subsequent repair costs. Both walkways that border the channel, one to the north and one to the south, are affected by such flooding. It is estimated by arboretum staff that the repair of each path requires approximately $400 in material costs and 2-3 person-days of labor in the event of a 24 hour storm with a 10 year return interval, such as the one on June 30th, 2013. At an average rate of $35 per hour of labor, including both laborer and management rates, the cost of walkway repair is estimated to be approximately $2,480 per 10 year 24 hour storm. It should be noted that the current Facilities Services labor rate for this type of work is around $40 per hour, but the arboretum uses student work study labor so the hourly rate is expected to be somewhat reduced. According to Sally Hoyt, the labor required to remove dislodged sediment from downstream areas such as roads, gutters, channels, and inlets is comparable to that of walkway repair, bringing the total cost of flood damages to about $4,200.

However, the value of benefits that accrue in the future is not directly comparable to capital costs paid in the present because money loses value over time, or in other words the value of today's money is discounted as time passes. In order to determine if the maintenance and repair benefits of the proposed channel redesign would outweigh the costs of implementation, the Present Value of future benefits was calculated using the following equation:

$$PV = \frac{FV}{(1+i)^n}$$

Where:
PV = Present Value
FV = Future Value
i = Discount Rate
n = Number of years from Present

A discount rate of 2% was used for the calculation as suggested by Sally Hoyt. The last storm event to damage arboretum walkways was in 2013 and was classified as a 10 year recurrence interval. Therefore, it was assumed that repairs would be needed every 10 years, with the first repair occurring in 2023. It should be noted that this is an approximate analysis because there is no guarantee that the 10 year storm will occur every 10 years to the year. The 10 year return interval simply means that, statistically speaking, a storm with that intensity has a 10% annual chance of occurring. Also, heavier, and therefore rarer, storms than the 10 year storm are not taken into account. With the available data, it would be difficult to estimate the additional flood damages associated with higher magnitude storms, and the effect of the channel redesign on such flood damages would be unclear and prohibitively hypothetical. Although the channel is technically designed to handle the 25 year storm, there is no freeboard criterion to act as a factor of safety and ensure flood damage reduction. Figure 22 shows the present value of project benefits as a function of the number of years from the present.

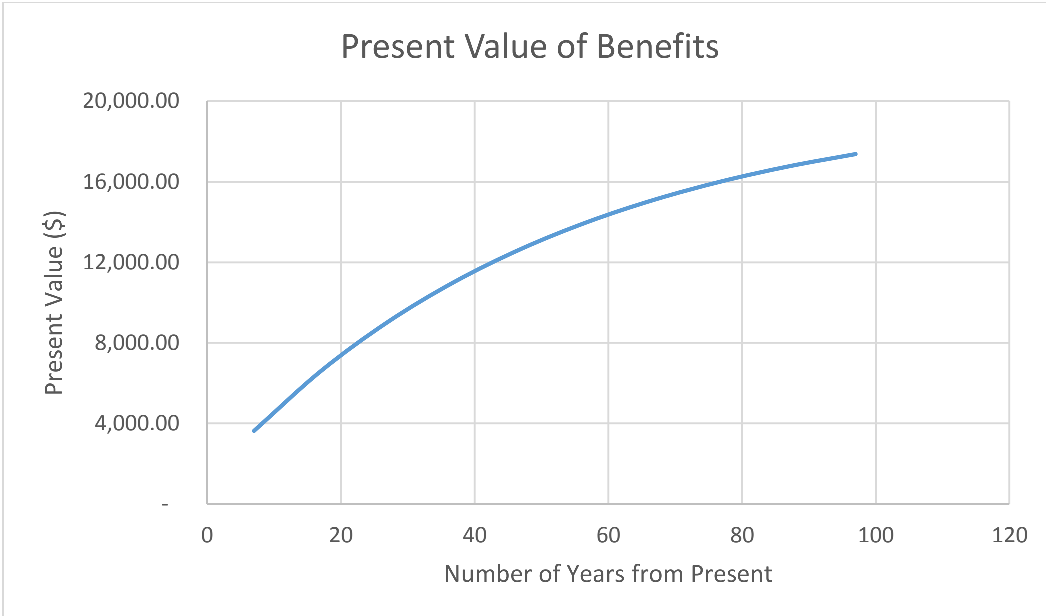

Figure 22: Present Value of Benefits as a function of Years from the present

Under these assumptions, the project will not break even within 100 years of implementation if only walkway repair benefits are considered. As summarized in Table 14, with the net present value of benefits estimated to be about $17,400, only about 65% of capital costs would be recovered and the Net Present Value (NPV) of the project would have a deficit of over $9,000. A sensitivity analysis on discount rates is depicted graphically in Figure 23 along with capital costs, which remain constant because they are paid in the present. Discount rates ranging from 0 – 6% were included in the analysis. As seen in figure 23, the costs of implementation surpass benefits only at interest rates lower than 1% and NPV remains negative.

Capital Costs	$	(26,500)
Avg. Repair Benefits, FV	$	420.00
Discount Rate		2.00%
Number of years		100
PV Repair Benefits	$	17,400
Net Present Value	**$**	**(9,100)**

Table 14: Figures Used to Calculate Net Present Value

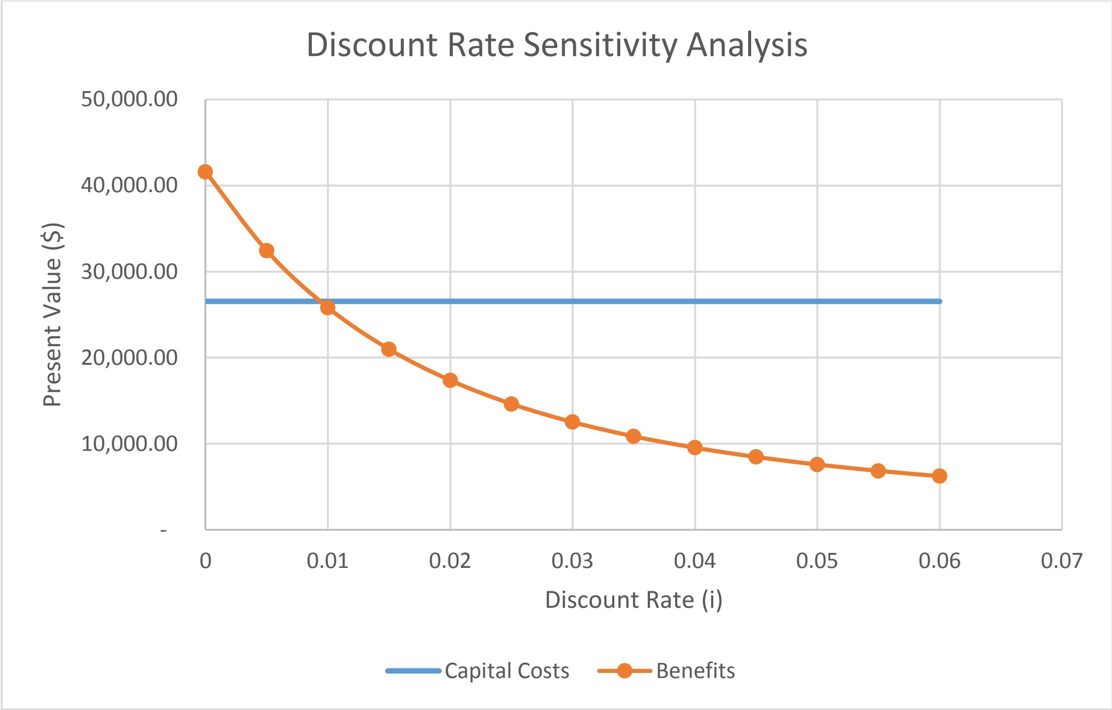

Figure 23: Graphic Depicting Discount Rate Sensitivity Analysis, Comparing Present Value of Benefits to Capital Costs at Various Discount Rates

CHAPTER 6: CONCLUSIONS

The analyses presented in Chapters 4 and 5 could not identify an economically attractive option for eliminating flood events during a 24 hour duration storm event with a 10 year recurrence interval and thus complying with the Town of Chapel Hill Design Standards. A lack of easily quantifiable benefits led to the conclusion that project implementation would come with a negative NPV of over $9,000. However, according to Sally Hoyt, the fact that the project will not break even in the 100 year timeframe does not necessarily mean that it will not be considered a viable project. The negative NPV will be presented for consideration during the review process and there may be additional factors to consider that are not as easily monetized.

For instance, the Coker Arboretum is within the Jordan Lake watershed and is thus subject to the Jordan Lake Rules laid out in order to reduce sediment and nutrient loads on the major drinking water reservoir, with major concern over nitrogen loads. As previously mentioned, trail washout leads to large amounts of sediment running off into Raleigh Street and into the drainage system. Aside from the ecological benefit of a reduced sediment load, an assessment could be undertaken to determine if a substantial amount of particulate nitrogen is retained by the walkways via sedimentation and infiltration of overland flow, and is thus transported downstream during flood events. The ESD has reviewed projects with nitrogen reduction costs ranging anywhere from $4,000/lb/yr - $40,000/lb/yr (Sierks, 2015). Evidence that limiting walkway sediment runoff would reduce particulate nitrogen loads along with sediment loads could increase the economic appeal of the project.

Similarly, arboretum management and the ESD could consider installing a constructed wetland or similar flood mitigating BMP. Further exploration and analysis of such BMPs could allow for easy integration into the arboretum's natural environment while alleviating flood problems with the added incentive of nutrient load reduction. Overall, with the application of the Jordan Lake Rules, stormwater control and nutrient reduction should be examined together. However, nutrient reduction was outside the scope of this study.

Additionally, it should be determined how much value is to be placed on design standard compliance. According to the Town of Chapel Hill Design Manual drainage facility design standards, stormwater infrastructure in the vicinity of local streets should be able to safely and effectively receive, convey, and discharge stormwater runoff resulting from the 10 year SCS – Type II design storm with a 24 hour duration, and the 25 year storm should be used as a check storm. This is not the case with the Coker Arboretum drainage channel. Further emphasis is placed on the criteria that streets will not be flooded nor curbs overtopped as a result of poor drainage infrastructure. It is unclear from the PCSWMM model or from site photographs whether this occurred, but it is clear that a significant amount of floodwater was conveyed into Raleigh Street due to channel overtopping on June 30th, 2013. The manual also states that existing infrastructure may be exempt, and the arboretum channel likely is, but the design standards exist to help maintain a clean and safe environment and minimize public nuisance.

Lastly, a more comprehensive flood damage study should be conducted before a course of action is selected. Historical rainfall data should be compared to storm frequency intervals and repair records in order to get a better idea of repair benefits. By reviewing arboretum repair records, it could be determined how much has been spent on flood repairs as a result of various storm events. Historical rainfall data could then be analyzed to determine the return interval of each storm that caused the need for flood repairs using the methods described in Chapter 2. Then repair benefits could be interpolated over the lifespan of the project. The PCSWMM model suggests that certain sections of the current channel will overtop their banks even during a storm event with a 5 year return interval. Furthermore, repairs will likely be more significant resulting from the 25 year storm than from the 10 year storm, but this is difficult to quantify because there is no available freeboard to act as a factor of safety and ensure flood damage reduction. However, it is likely that additional repair benefits exist other than those presented in Chapter 5.

APPENDIX A: RAINFALL DATA

Duration	Rainfall (in)	Ending Time	Data Used	Frequency per NOAA Atlas 14
5-minute	0.47	6/30/2013 3:02	KIGX All	2-year
10-minute	0.77	6/30/2013 3:02	KIGX All	2-year
15-minute	1.00	6/30/2013 3:02	KIGX All	2-year
30-minute	1.61	6/30/2013 14:25	KIGX All	5-year
1-hour	2.09	6/30/2013 14:56	KIGX Hourly	5-year
2-hour	3.07	6/30/2013 14:56	KIGX Hourly	10-year
6-hour	3.07	6/30/2013 14:56	KIGX Hourly	5-year
12-hour	4.25	6/30/2013 14:56	KIGX Hourly	10-year
24-hour	4.87	6/30/2013 14:56	KIGX Hourly	10-year
2-day	6.70	6/30/2013 14:56	KIGX Hourly	10-year
3-day	6.72	6/30/2013 14:56	KIGX Hourly	10-year to 25-year
4-day	7.41	6/30/2013 14:56	KIGX Hourly	25-year
7-day	7.55	6/30/2013 14:56	KIGX Hourly	10-year
10-day	8.52	6/30/2013 14:56	KIGX Hourly	10-year
20-day	9.68	6/30/2013 14:56	KIGX Hourly	5-year
30-day	15.07	6/30/2013 14:56	KIGX Hourly	25-year
45-day	18.54	6/30/2013 14:56	KIGX Hourly	50-year
60-day	18.95	6/30/2013 14:56	KIGX Hourly	10-year

Table 15: Real-time Rainfall Data and Frequency Estimates for the 6/30/2013 Storm Event (Hoyt, 2014)

PRECIPITATION FREQUENCY ESTIMATES (in inches)							
	Average Recurrence Interval (years)						
Duration	1	2	5	10	25	50	100
5-min:	0.41	0.48	0.56	0.61	0.68	0.72	0.76
10-min:	0.66	0.77	0.89	0.98	1.08	1.14	1.2
15-min:	0.82	0.97	1.13	1.25	1.36	1.45	1.52
30-min:	1.12	1.34	1.6	1.8	2.02	2.18	2.33
60-min:	1.4	1.69	2.06	2.35	2.69	2.95	3.2
2-hr:	1.68	2.02	2.49	2.87	3.33	3.7	4.05
3-hr:	1.79	2.16	2.66	3.08	3.61	4.04	4.46
6-hr:	2.15	2.59	3.2	3.71	4.37	4.92	5.47
12-hr:	2.54	3.06	3.8	4.44	5.28	5.99	6.71
24-hr:	2.96	3.58	4.47	5.17	6.11	6.86	7.62
2-day:	3.46	4.17	5.17	5.95	6.99	7.81	8.64
3-day:	3.67	4.41	5.44	6.25	7.33	8.19	9.07
4-day:	3.87	4.64	5.71	6.54	7.68	8.57	9.49
7-day:	4.44	5.3	6.44	7.34	8.57	9.54	10.53
10-day:	5.05	6	7.21	8.15	9.42	10.43	11.44
20-day:	6.76	7.97	9.41	10.56	12.11	13.34	14.57
30-day:	8.39	9.88	11.47	12.72	14.36	15.62	16.87
45-day:	10.69	12.52	14.32	15.72	17.55	18.95	20.31
60-day:	12.84	14.97	16.89	18.37	20.28	21.72	23.11

Table 16: Tabulated NOAA Precipitation Frequency Estimates for Various Rainfall Durations (NOAA, 2014)

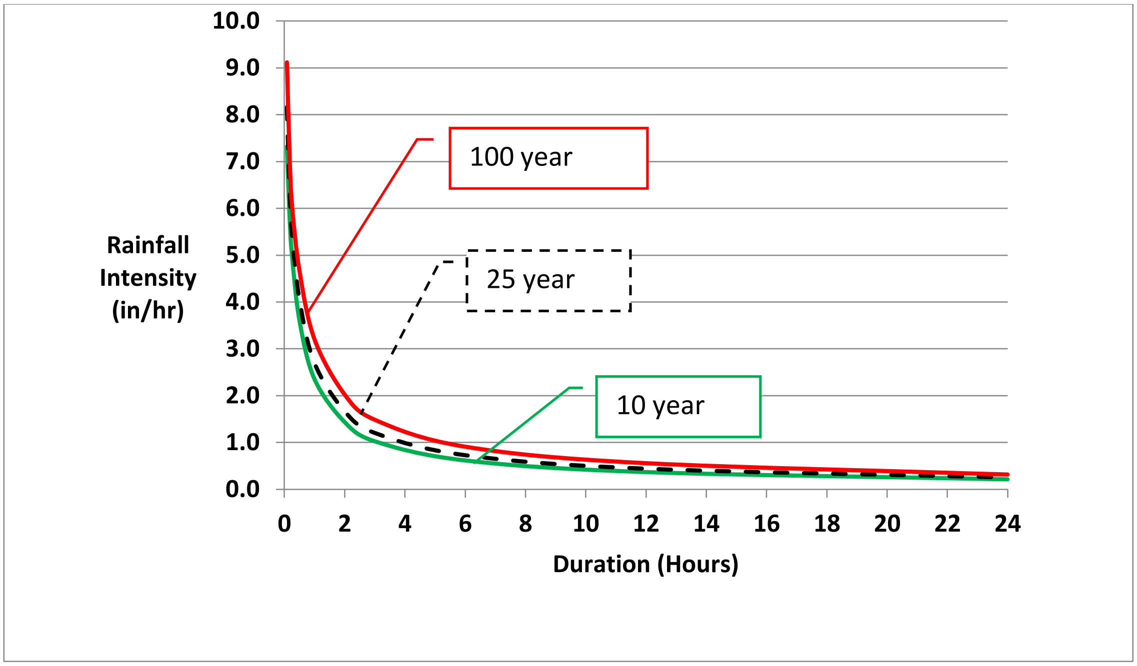

Figure 24: IDF Curves per NOAA Precipitation Frequency Estimates (Kolsky, 2015)

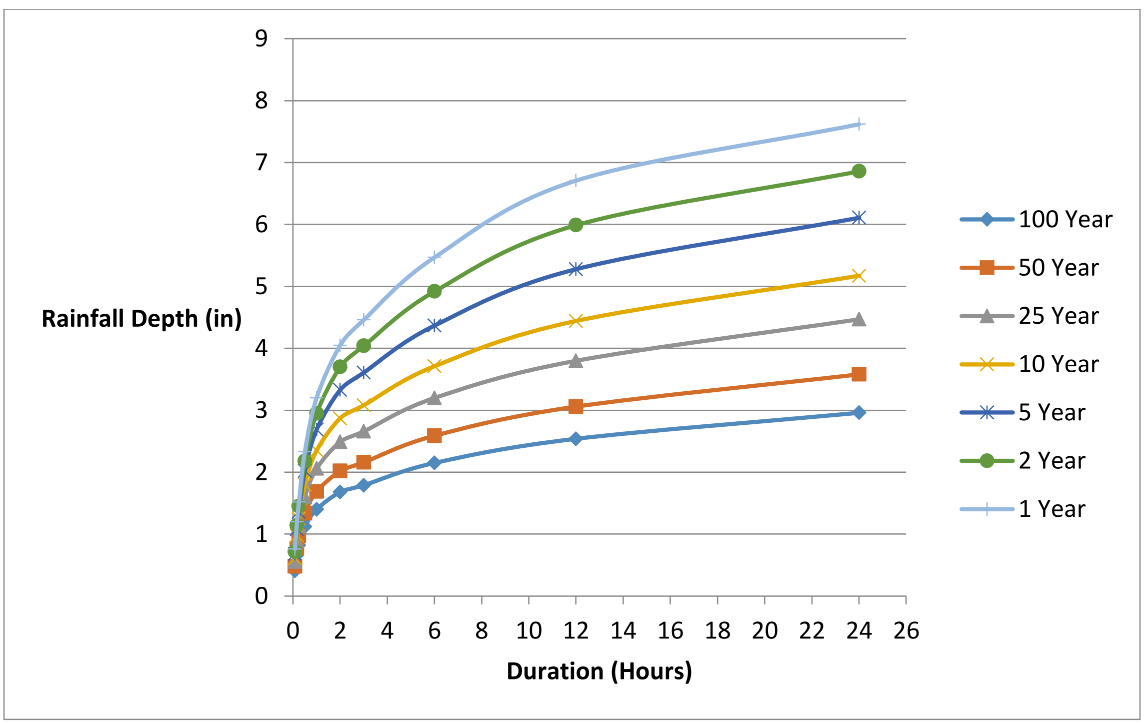

Figure 25: Rainfall Depth-Duration-Frequency Curves per NOAA Precipitation Frequency Estimates (Kolsky, 2015)

APPENDIX B: PROJECT CALCULATIONS

Time of Concentration (T_c)

Subcatchment time of concentration was calculated using the kinematic wave formulation:

$$T_c = \left(\frac{L}{a * i^{*(m-1)}}\right)^{\frac{1}{m}}$$

Where:

T_c = time of concentration in seconds
L = subcatchment length in feet
i^* = rainfall intensity in ft/s
a,m = kinematic wave parameters

For Manning's equation:

m = 5/3
a = $(1.49/n)*S^{1/2}$

Where:

n = Manning's roughness for overland flow
S = subcatchment slope

10 yr 24 hr i^* (in/hr)	0.215
10 yr 24 hr i^* (ft/s)	4.98E-06

Subcatchment	Area (acres)	Area (ft^2)	Width (ft)	Length (ft)	% Impervious	% Pervious	Impervious n	Pervious n	Slope (ft/ft)	a	Denominator	t_c (sec)	t_c (min)
BATTLE-18	3.58	155945	125	1248	43.27	56.73	0.011	0.24	0.043	12.9	0.0038	2053	34.2
BATTLE-19	1.35	58806	558	105	59.57	40.43	0.011	0.24	0.041	16.8	0.0049	398	6.6
BATTLE-20	1.56	67954	593	115	60.38	39.62	0.011	0.24	0.055	19.7	0.0057	381	6.3
BATTLE-21	2.3	100188	624	161	43.07	56.93	0.011	0.24	0.035	11.6	0.0034	641	10.7

Table 17: Time of Concentration Calculations

Steady State Discharge Capacity

The steady state discharge capacity of the channel cross-sections was calculated using Manning's equation and the continuity equation.

Manning's equation:

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

Where:

v = water velocity
n = Manning's roughness
R = hydraulic radius, ft
S = channel slope, ft/ft

Continuity equation:

$$Q = A * v$$

Where:

Q = flow rate, ft^3/s
A = cross-sectional area
V = water velocity

Existing Conditions							
Cross Section #	Area (ft^2)	Wetted Perimeter (ft)	Hydraulic Radius	Manning's n	Slope (ft/ft)	Velocity (ft/s)	Discharge Capacity (cfs)
1	2.88	4.92	0.59	0.035	0.022	4.42	12.73
2	1.9	3.92	0.48	0.035	0.022	3.90	7.40
3	6.36	6.73	0.95	0.035	0.016	5.19	32.98
4	2.23	4.05	0.55	0.035	0.016	3.62	8.07
5	4.54	5.83	0.78	0.035	0.016	4.56	20.69
6	4.1	5.46	0.75	0.035	0.016	4.45	18.24

Table 18: Existing Conditions Steady State Discharge Capacity Calculations

Alternative Design Options (10 yr)												
Cross-Section #	Base (ft)	Depth (ft)	Side Slope (rise:run)	Area (ft^2)	Wetted Perimeter (ft)	Hydraulic Radius	Manning's n	Slope (ft/ft)	Velocity (ft/s)	Discharge Capacity (cfs)	10 yr 24 hr Peak Flow (cfs)	25 yr 24 hr Peak Flow (cfs)
3 (w/o freeboard)	2.3	1.61	1	6.28	6.85	0.92	0.035	0.016	5.08	31.95	31.95	45.02
4 (w/ freeboard)	2.3	1.92	1	8.10	7.73	1.05	0.035	0.016	5.56	45.02	31.95	44.71
5 (w/ freeboard)	2.3	1.92	1	8.10	7.73	1.05	0.035	0.016	5.56	45.02	31.95	44.71
6 (w/ freeboard)	2.3	1.92	1	8.10	7.73	1.05	0.035	0.016	5.56	45.02	31.95	44.71

Table 19: Channel Redesign Options to Increase Discharge Capacity
(Note that cross-section #3 does not include the freeboard criterion for the sake of comparison)

APPENDIX C: TABLES AND FIGURES USEFUL FOR OVERLAND FLOW ROUTING

Surface	n
Smooth asphalt	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Good wood	0.014
Brick with cement mortar	0.014
Vitrified clay	0.015
Cast iron	0.015
Corrugated metal pipes	0.024
Cement rubble surface	0.024
Fallow soils (no residue)	0.05
Cultivated soils Residue cover < 20% Residue cover > 20%	 0.06 0.17
Range (natural)	0.13
Grass Short, prarie Dense Bermuda grass	 0.15 0.24 0.41
Woods Light underbrush Dense underbrush	 0.40 0.80

Source: McCuen, R. et al. (1996), *Hydrology,* FHWA-SA-96-067, Federal Highway Administration, Washington, DC

Figure 26: Manning's Roughness for Overland Flow

Impervious surfaces	0.05 - 0.10 inches
Lawns	0.10 - 0.20 inches
Pasture	0.20 inches
Forest litter	0.30 inches

Source: ASCE, (1992). *Design & Construction of Urban Stormwater Management Systems*, New York, NY.

Figure 27: Typical Values for Depression Storage by Land Cover Type

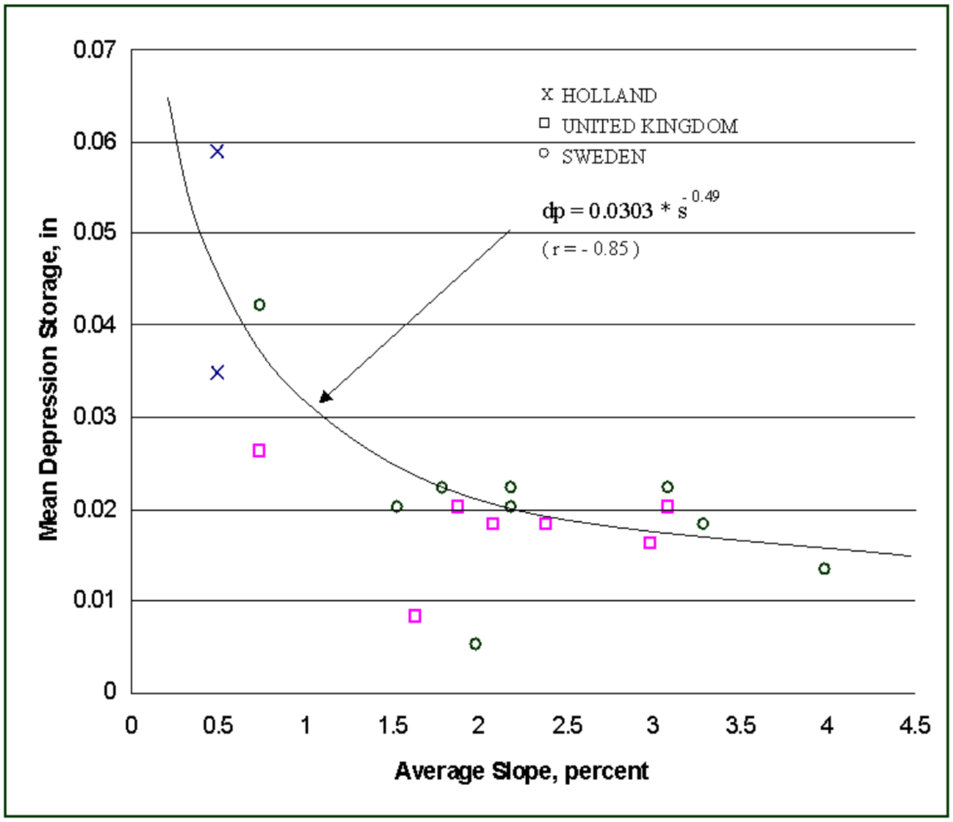

Figure 28: Mean Depression Storage as a Function of Catchment Slope, Guidance for SWMM Parameter Selection for Overland Flow Routing Calculation

APPENDIX D: CALCULATIONS FOR COSTING PARAMETER FORMULATION

Per Cross Section				**Total**
XS #	3	5	6	
Length[1]	50	50	50	150
Demolition				
Width[2] (ft)	3.33	2.33	2.58	
Depth[3] (ft)	0.5	0.5	0.5	
CF	83	58	65	206
CY	3	2	2	8*
Excavation				
Area[4] (ft)	1.22	2.24	2.05	
CF	61	112	103	276
CY	2	4	4	10
Grading				
Width[5] (ft)	7.73	7.73	7.73	
SF	387	387	387	1160
SY	43	43	43	129
Concrete Lining				
Width[2] (ft)	3.8	3.8	3.8	
SF	190	190	190	570
Coir Fiber Matting				
Width[6] (ft)	12	12	12	
SF	600	600	600	1800
SY	67	67	67	200
Temporary Seeding				
Width[6] (ft)	40	40	40	
SF	2000	2000	2000	6000
AC	0.05	0.05	0.05	0.14

Table 20: Costing Parameters, with Figures under Each Category Calculated per Cross-Section then Summed to Obtain Channel-wide Estimates (total is rounded to the nearest whole number)*

Notes:

1	Each cross section was assumed to represent an equal length of channel
2	Existing stone lining was assumed to cover channel bed and 9" up either bank on average
3	Stone lining assumed to be 6" thick
4	Difference between existing and proposed cross-sectional area
5	Grading assumed for all of channel bed and banks
6	Both banks are accounted for

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