RENOVATUS SOLUTIONS
more than engineers

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Mr. Joseph T. Hoke, Jr., P.E.
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Dear Mr. Hoke,

On behalf of the student company Renovatus Solutions, I submit herewith a report entitled "Cashiers, North Carolina Stormwater Study and Flooding Recommendations" to be reviewed by the US Army Corps of Engineers Savannah District and Jackson County North Carolina. This report serves as a preliminary engineering design and evaluation with the intent to mitigate flooding in Cashiers, North Carolina.

Your time investing in the Georgia Institute of Technology and our student company is greatly appreciated.

Sincerely,

Rehal Kharel

Rehal Kharel, Project Manager

April, 2015

Cashiers, North Carolina Stormwater Study and Flooding Recommendations

A Capstone Design Project Report

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Executive Summary

The Savannah District U.S. Army Corps of Engineers and Jackson County North Carolina have requested preliminary engineering design recommendations and evaluations from Renovatus Solutions in order to plan and ultimately implement a system to help mitigate flooding in Cashiers, North Carolina. Cashiers is located just north of the Georgia-North Carolina boarder in the southern Blue Ridge Mountains. Scenic landscapes, close proximity to hiking and nature trails, and a mild climate attract a large number of tourists and seasonal residents every summer. A site visit report provided by Savannah District, a site visit conducted by Renovatus Solutions, and an additional requested topographic survey affirmed the extent of the flooding issues.

Based from these observations and additional research we determined that the root cause of the flooding is the prevalence of poorly drained Nikwasi soil in the area, especially in the wetland region south of Frank Allen Rd. With this conclusion, a two part solution framework was developed to address the flooding issues in light of Nikwasi soil: 1) Dredging the channel upstream and downstream of the Frank Allen Rd culverts to ease the passage of water through the channel, allowing more water to leave the flood impact area and thus draw down the standing water. 2) Prevention of further flood events through the implementation of sedimentation/detention basins upstream of the Frank Allen Rd culverts.

The intent of this preliminary report is to present this two part solution in a way that addresses the objectives delineated by the sponsor of the project, Savannah District USACE. The solutions presented in the report are designed to be effective on the immediate and long-term time scale while maintaining reasonable cost in order to drive further community decision-making to begin addressing the flooding issues.

In order to supplement detention basin design, we performed a stormwater conveyance analysis and used the NCDENR BMP Stormwater Manual to determine appropriate best management practices. Two options were proposed; a single basin option with a single wet detention basin and a double basin option composed of a dry detention basin and a wet detention basin. Both options offered advantages and disadvantages.

While it does not address the flooding issues, Savannah District required us to investigate roadbed damage, estimate repair costs frequency, and quantify economic impacts due to the overtopping of the road. It was found that performing the rout and seal repair method while closing the road for repairs was the most effective and least expensive solution.

We present as our final recommendations to address the flooding a combination of stormwater detention basins, sediment removal, culvert maintenance, and roadway repairs. We recommend implementing all of these components to achieve the best results of flooding mitigation. Leaving the flooding issues in Cashiers unaddressed is unadvisable as this poses unnecessary risk to the community.

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1. Introduction

Renovatus Solutions was retained by the Savanah District US Army Corps of Engineers and Jackson County to provide recommendations to help mitigate flooding in Cashiers, North Carolina. The city is located in a relatively flat valley within the southern Blue Ridge Mountains. The flood impact area is located just north of Cashiers Lake and the wetlands leading into the lake (Figure 1). Cashiers receives an average of 85 inches of rain per year, which is very high when compared to the rest of the United States (Current Results Nexus 2015). Flooding of Frank Allen Road occurs at less than 1 inch of rain (Gerald Green, personal communication, Jan 2015).

Figure 1. Cashiers Flood Impact Area (source: Google Earth)

Cashiers receives most of its rain storms during the summer months. Also during the summer, the population of Cashiers increases dramatically with the influx of seasonal residents. The city is a hub for vacation homes due to the scenic mountain environment, close proximity to hiking trails, and moderate summer temperatures. Flooding of Frank Allen Road in addition to elevated traffic from seasonal tourism is a significant problem for the locals and businesses of Cashiers.

Frank Allen Road is located just north of a wetland area leading into Cashiers Lake. Cashiers Creek flows under the road through two 81" by 59" arch pipe culverts. During flooding events, water backs up through the culverts and overtops the road. This makes it difficult for locals and tourists to access businesses on Frank Allen Road including restaurants, a local paper, a volunteer fire department, a post office, realtor groups, and recreational areas. Access to these businesses is particularly important given the number of tourists Cashiers receives each year.

An earthen dam was constructed pre-World War II to form Cashiers Lake. The dam was remediated in 2012 to address issues including trees and brush growing on the dam embankments, leakage from a culvert spillway, no working bottom drain, and an inadequate single channel spillway (Christopher Capellini, unpublished dam repair report, June 2011). A second spillway was added to increase capacity in storm conditions. Flooding had been occurring in the area before the dam was renovated, and the water table remained high around Frank Allen Road when the lake was drained for construction.

Renovatus was provided with a site visit report from the Savanah District Army Corps of Engineers. The site visit was conducted by the Army Corps and Jackson County and potential causes of flooding were identified. The report indicates a high water table near Frank Allen Road, sedimentation of Cashiers Creek north and south of Frank Allen, and heavy siltation in several culverts and ditches within the flood impact area (Joe Hoke, unpublished survey report, September 2014). The report concluded that the flatter area creates slower flows of the river resulting in increased sedimentation (Joe Hoke, unpublished survey report, September 2014). These slower flows, in addition to increased sediments from runoff due to development in the area, result in a higher water table and a backwater effect when it rains (Joe Hoke, unpublished survey report, September 2014).

With this information, along with an additional site visit and survey of the area, Renovatus was able to analyze the storm water system at Cashiers and develop several recommendations.

1.1 Problem Statement

Periodic flooding has been occurring throughout the stormwater system of Cashiers, NC. Flooding is most apparent along Frank Allen Road, where the water significantly overtops a section of the road. The problem can be attributed to factors in the areas both north and south of Frank Allen Road.

North of Frank Allen Road:

1. Flooding along Frank Allen Road:
   Water consistently overtops the lowest point of Frank Allen Road during rain events as low as 1 inch causing potential hazards to cars and pedestrians as well as contributing to long term roadbed damage.
2. Significant sediment in runoff along US 64:
   US 64 does not contain appropriate BMPs to channel and trap sediment runoff from the highway allowing sediment to freely enter the stormwater system
3. Siltation of stormwater channels:
   Streams and stormwater channels are heavily silted due to accumulation of sediment and lack of maintenance over time
4. Siltation of stormwater pipes:
   Stormwater pipes and culverts are heavily silted due to accumulation of sediment lack of maintenance over time
5. Unpaved features contributing fine sands into stormwater system:
   Prevalence of gravel parking lots, dirt roads, and unpaved areas (Figure 2) is believed to be contributing to sedimentation
6. Limited detention ponds and sediment traps:
   A limited number of sedimentation basins and BMPs are in place to alleviate sedimentation and detain excess stormwater.

Figure 2. Unpaved Features in Cashiers.

South of Frank Allen Road:

The wetland area north of Cashiers Lake and directly south of Frank Allen Road have accumulated sediments over time. The accumulation has led to the expansion of the wetland area as seen in Figure 3 below. This sedimentation impedes the flow of stormwater into Cashiers Lake. In addition to the sedimentation, the soil characteristics also impedes flow through the wetland area. The Cashiers Creek Wetland is made up of Nikwasi soil which is a poorly drained soil. The soil retains large amounts of water, obstructing flow downstream. Sediment accumulation, along with Nikwasi soil, can be attributed to high volume of sediments upstream of the wetland area.

Figure 3. Wetland area increase**.** Aerial View of Cashiers Lake and wetland area in 1995, 2005, and 2014 (left to right)

1.2 Site Description

Cashiers is a town with a total area of 1.1 sq.-mile located at **35°6′43″N 83°5′58″W** in Jackson County, North Carolina. It is situated at an elevation of 3,487 feet and has an official population of 1,974 and a visiting population of 10,000-15,000 during the summer season. It receives heavy rainfall with an annual rainfall of approximately 80-100 inches in the recent years. The town is surrounded by mountains which range up to 5,127 feet. Figure 4 displays a land use map of Cashiers; Table 1 displays the area attributed to each land use type.

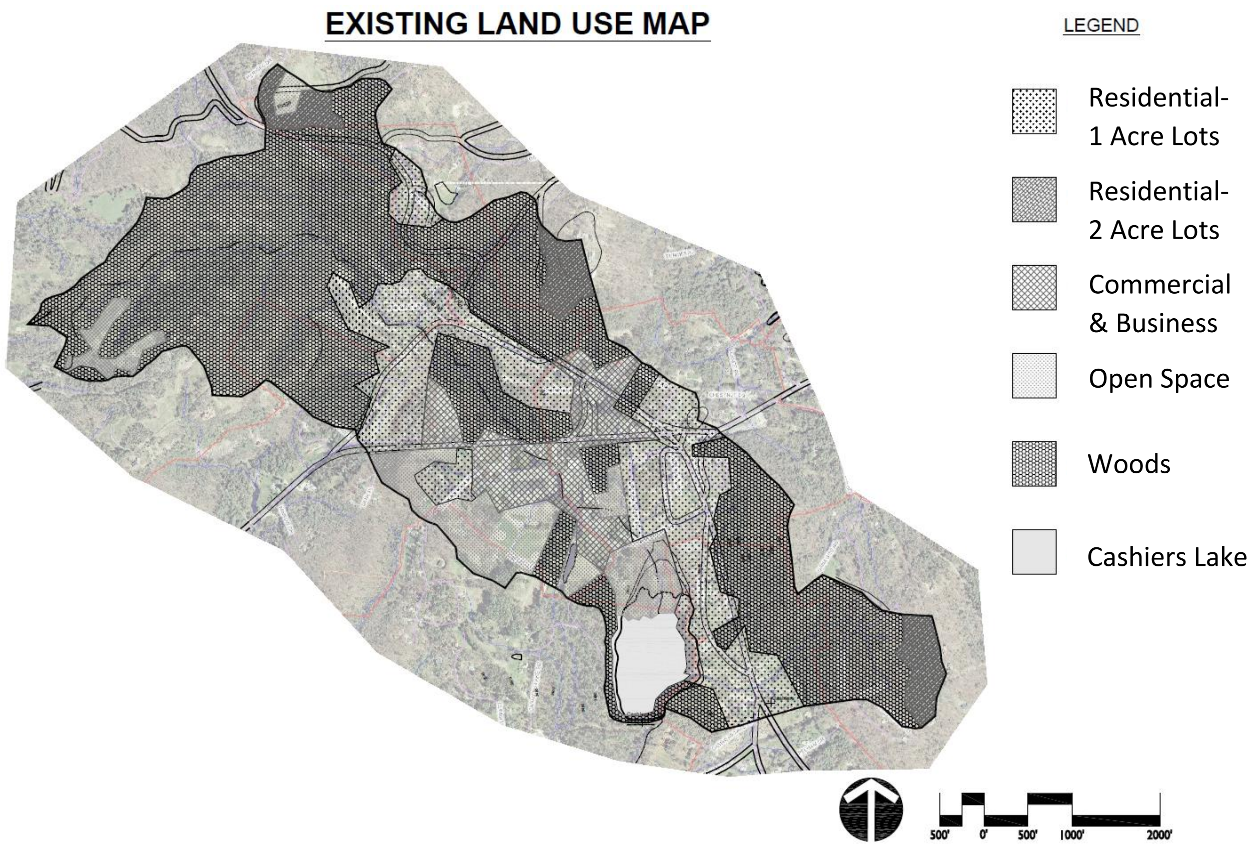

Figure 4. Cashiers Land Use Map.
This map depicts the post-development condition of land use in the watershed as given in the dam report (see Appendix C).

Table 1. Cashiers Lake Watershed Land Use Areas.

Surface Description	Area (Acres)
Water	24
Residential 2 acre	42
Residential 1 acre	137
Commercial and Business	78
Open Space (Good)	36
Woods (Good)	351
Total Area	671

Cashiers Lake Watershed

The watershed outlet is considered to be located at Frank Allen Road by the pair of culverts. The watershed study area is approximately 480 acres and not 671 acres as shown in the Dam Report. This is because the culverts at Frank Allen Road are depicted as the outlet of the watershed and anything south of Frank Allen Road is disregarded in the stormwater study. The flood impact area is shown in Figure 5 below along with some key locations denoted by letters.

Figure 5. Cashiers Key Locations.

Description of key locations in Cashiers.

For the purposes of identifying the key locations at Cashiers, the flood impact area has been divided into the East and West side. The streams from the west side are all connected to the east near Frank Allen Road and are directed downstream by culverts into the wetland and the reservoir.

A. Highway 64: Most of the study is done south of Highway 64. It is important to establish this boundary to isolate the flood impact area. The streams flow south of Highway 64 eventually into Cashiers Lake. The area above the highway have much higher elevation, therefore all of the streams flow downwards and sediments begin settling around these areas.

B. Village Green Wetlands: This wetland is right below Highway 64 and has numerous streams from both east and the west sides. It is approximately 5 acres in area.

C. Frank Allen Road: This road is right below the Wetland (North) and is the most affected by flooding and sedimentation. Under this road lies a pair of culvert

(CMP59) which directs streams from both the east and west side into the wetland (south) and finally into the Cashiers Lake.

D. **Cashiers Creek:** This is a name given to the stream that collects all the small streams from the west and flows under the Frank Allen Rd. into the wetland.

E. **Cashiers Lake Wetlands:** This is where all of the streams from north of Frank Allen Rd. is conveyed. The streams along with its sediments are trapped in this wetland which is connected and drained into the reservoir (Cashiers Lake). This wetland is approximately 13.11 acres in area.

F. **Cashiers Lake:** This is a recreational lake which is privately owned by the Canoe Club. This reservoir serves as the collection point for most of the water and sediments from the north. Cashiers Lake approximately covers 7.5 acres in area and is impounded by an earthen dam.

G. **Cashiers Lake Dam:** This dam was constructed pre-1940's but was recently remediated to address structural deficiencies in the dam embankment. During restoration, an additional spillway was added to increase the capacity for high-flow conditions. Water from Cashiers Lake flows into the Chattooga River.

1.3 Site Visit Report

The main objective of our site visit was to familiarize ourselves with the flood impact area, primarily through visual surveys. Within the flood impact area, several locations were identified as key areas to investigate:

- Frank Allen Rd double culverts
- The channel leading to the Frank Allen culverts
- Frank Allen roadway over the double culverts
- Roadside sediment depositions along Route 64

Some locations outside of the flood impact area were also included as key areas due to their possible relation to the flooding and sedimentation:

- Cashiers Dam—thought to have raised the local water table, contributing to flooding

In addition to the visual surveys, obtaining samples of the sediments deposited in stream channels and along roadways was a main objective for the site visit.

1.3.1 Site Visit Observations

We determined from a brief interview with a local Cashiers resident that the last rainfall prior to our visit had occurred three to four days prior and no rainfall was observed during our visit. However, the water level at the culverts was high (Figure 6); there was approximately a foot and a half of clearance between the top of the culvert and the water surface. This observation suggests that the water level is either consistently high or drainage from the area occurs at a rate longer than three to four days.

Figure 6. Frank Allen Rd Double Culverts (one culvert from two different angles). For a sense of scale, the culverts are approximately five feet in height. In addition to the high water level, there were many aquatic plants growing near the culvert inlets.

The high water level in the culverts is believed to be due to a high water table in the area. Evidence of this was found in a depression in the ground surface nearby (Figure 7). This depression is located east of the culverts about 150 feet away. The water surface of this ponding was not connected to the water surface of the stream leading to the Frank Allen culverts. This ponding, along with the fact that it had not rained recently, indicates a high water table in the area.

Figure 7. Ponding Along Frank Allen Rd. About 150 ft east of the Frank Allen Rd culverts, the ponding water surface is not connected to the standing water at the culverts.

In addition to the evidence of a high water table, signs of past flood events were visible at Frank Allen Rd. Erosion of the road shoulder material within the area of ponding was observed. This type of erosion was not found in areas without ponding. It is possible that this erosion was caused by water scouring away at the roadbed during flood events. Furthermore, ponding in this location can be seen in Google Maps (Figure 8) on multiple occasions, indicating that this ponding is a recurring phenomenon.

Figure 8. 2012 Frank Allen Rd Ponding. Google Earth image showing ponding along Frank Allen Rd in relation to the Frank Allen Rd culverts. Scale bar is located in bottom left.

Further evidence of past water flow over Frank Allen Road was observed across the road from the ponding. Vegetation on this side of the road was bent and flattened towards the south in the same direction that water would flow over the road (Figure 9).

Figure 9. Evidence of Overtopping of Frank Allen Rd. Flatted plant vegetation as evidence of past high water flow conditions. Photo taken on Frank Allen Rd, facing southeast.

Summary of Observations:

1) The water table in the vicinity of the Frank Allen culverts is high. Frank Allen Rd has been overtopped by floodwaters, likely multiple times in the past.
2) The Frank Allen culverts receive flow from western, northern, and eastern areas of Cashiers.
3) There is an abundance of loose sediment surfaces.

1.4 Cashiers Topographic Survey

Based on the site visit discussed in the previous section (section 1.3.1), we requested a topographic survey to capture the following items:

1) Elevation of the flood impact area at a resolution of one foot contour intervals
2) Centerlines of roads and streams
3) Dirt or gravel surfaces within the flood impact area

Through the provided topographic survey, such details as the water surface elevations, stream centerlines, stormwater infrastructure, etc. provided us with necessary components for our analyses. The topographic data also helped to site the potential locations of the proposed detention basins.

The delivered topographic survey is included in Appendix A

2. Project Approach

Flooding and sedimentation issues in Cashiers are believed to be tied together. The large amounts of sediment released in the stormwater system from dirt and gravel surface features exacerbate the flooding issues by occupying space in the stormwater channels and culverts, causing water to back up and rise. If sediment removal is to be effective, the cause of the flooding must be addressed such that this cycle of sedimentation exacerbating flooding does not continue. In order to recommend appropriate solutions to the stormwater system, we sought to understand the cause of the flooding issues. Our findings lead us to believe that the root of the Frank Allen Rd flooding issues lies in the poorly drained soils in the area. The following lines of evidence have led to this conclusion:

1) **Flooding at the Frank Allen Rd culverts has been decoupled from the water level of Cashiers Lake.** During the initial stages of this study, we believed that the flooding issues were related to the downstream reservoir increasing the local water table. However, Cashiers Lake has been drawn down four feet for remediation on the dam embankment during the winter of 2011-2012 (Figure 10). A reduction in flooding events as well as standing water at the Frank Allen Rd. culverts was not apparent to observers during the time that the reservoir had been drawn down. If the high water table in the Frank Allen Rd. culverts is being caused by the presence of the Cashiers Lake Dam downstream, a decrease in the water level of the culverts during the time that the reservoir was drawn down would be expected. The fact that no documented decrease occurs suggests that the high water table is not directly caused by the level of the reservoir.

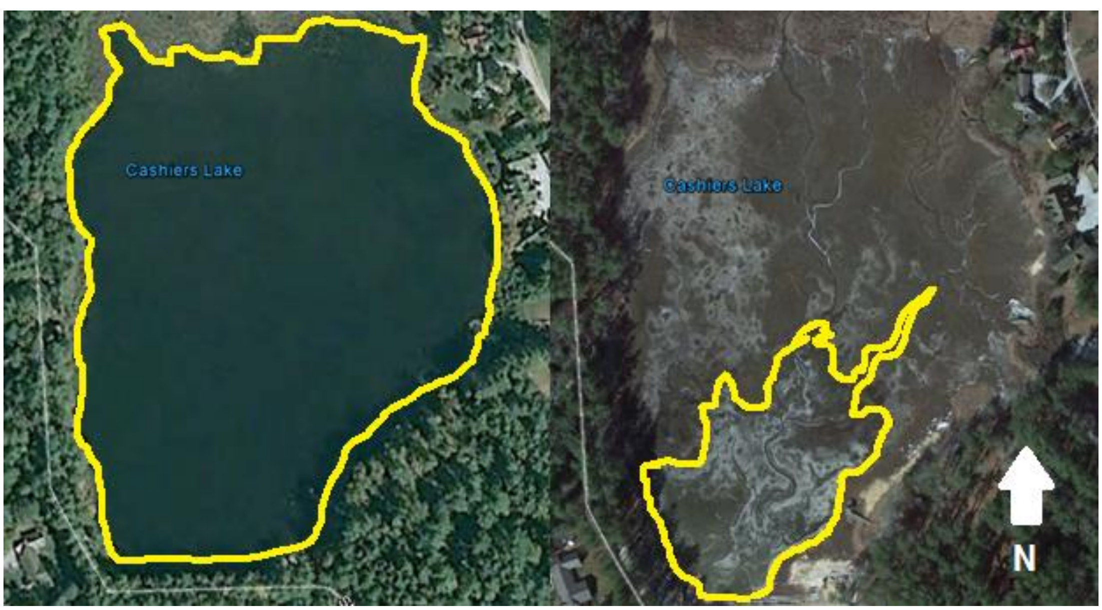

Figure 10. Cashiers Lake Water Surface Before and After Draw Down. The left image is dated 5/30/2009 (before dam remediation), and the right image is dated 2/11/2012 (during dam remediation).

2) **The soils in the region of flooding are conducive to ponding conditions.** If drawn down of Cashiers Lake did not produce a lower water table, there must be another mechanism of flooding. The report provided for the remediation of the Cashiers Lake Dam showed poorly draining soils in the region of high water table. Figure 11 provides a soils classification map of the area.

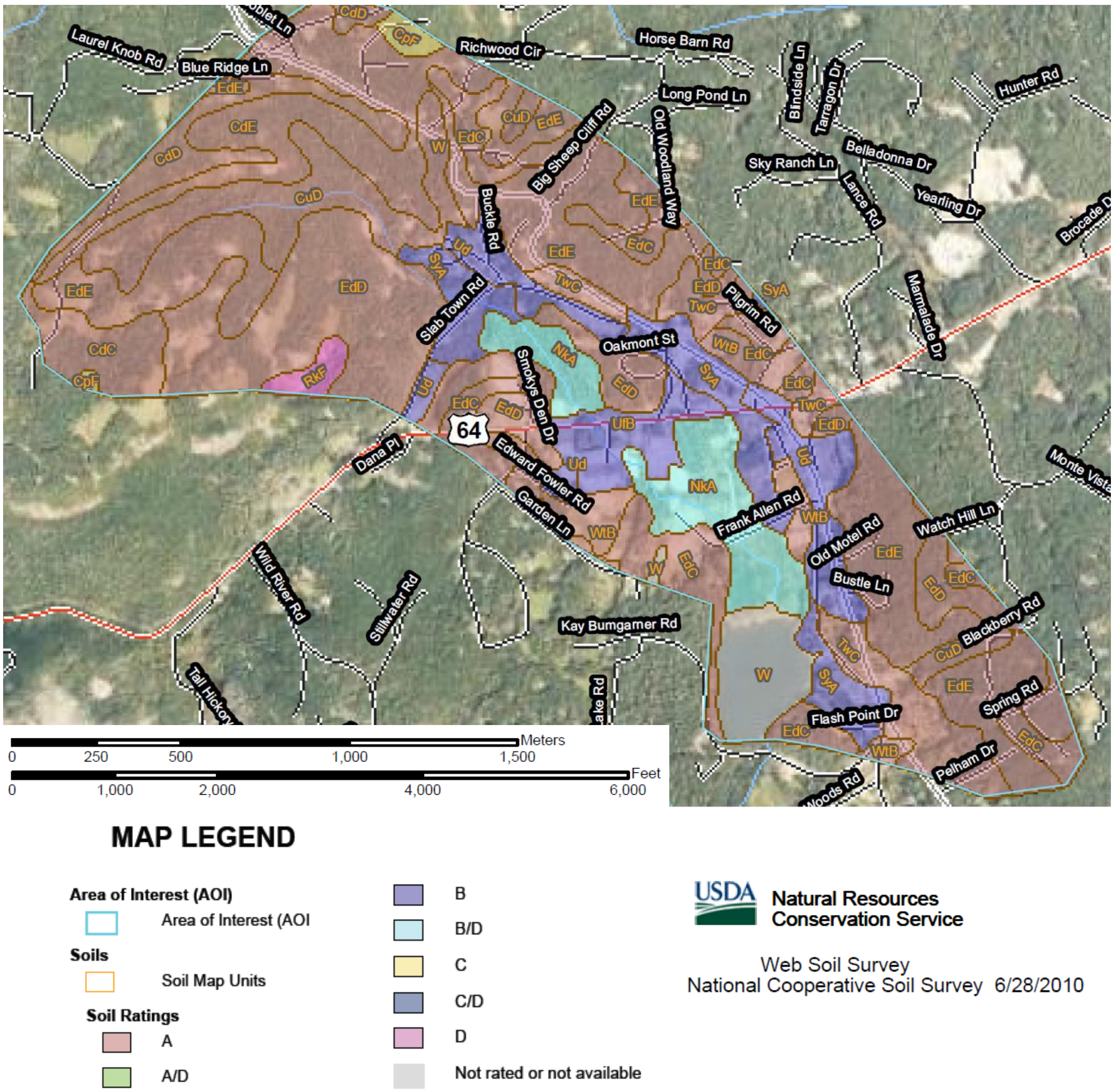

Figure 11. NRCS National Cooperative Soil Survey Map of Cashiers Lake Drainage Area. A and B soil ratings are well drained soils while B/D soils are poorly draining when wet. Take note that Frank Allen Rd is located within the B/D soils. Full size version can be found in Appendix B.

Figure 11 classifies the area in which Frank Allen Rd floods as Nikwasi soil. According to the Soil Survey of Jackson County, North Carolina (1997), Nikwasi soils are “nearly level, and poorly drained.” The soil survey goes on further to state that for the Nikwasi soils “the flooding, the wetness, the ponding, soil compaction, runoff from the higher adjacent areas, and damage to streambanks are serious management concerns” (Soil Survey of Jackson County, North Carolina 1997). This description of Nikwasi soils gives the impression that water entering the soils has a high chance of accumulating to build a high water table and potentially, standing water.

3) **Ponding due to the Nikwasi soils is leading to the elevated water table at Frank Allen Rd.** The soils maps in Figure 11 shows that Nikwasi soils lie just upstream of the reservoir. The fact that the soil type is characterized as poorly-draining and subject to flooding and ponding led us to believe that the Nikwasi soils just upstream of the reservoir are responsible for accumulating incoming water and retaining that water such that the local water table rises and presents standing water in areas of low elevation (i.e. the Frank Allen Rd culverts and along Frank Allen Rd).

The lines of evidence presented above lead us to the following conclusions:

- A solution involving lowering the water surface elevation of Cashiers Lake to increase the head differential between the water table at Frank Allen Rd culverts and Cashiers Lake would not have the desired effect of reducing the water table at Frank Allen Rd culverts. This was demonstrated during the drawdown of Cashiers Lake for the 2011 to 2012 dam remediation.
- The flow from Frank Allen Rd to Cashiers Lake is being retained somewhere between the two areas. We believe that the Nikwasi soils play a role in this retention of water.

With these conclusions, we developed the following two part solution framework to address the flooding issues in the vicinity of the Frank Allen Rd culverts:

1) Dredging the channel upstream and downstream of the Frank Allen Rd culverts to ease the passage of water through the channel, allowing more water to leave the flood impact area and thus draw down the standing water.
2) Prevention of further flood events through the implementation of sedimentation/detention basins upstream of the Frank Allen Rd culverts.

3. Project Objectives & Requirements

The sponsor of this project, the Savannah District of the US Army Corps of Engineers, delineated the following objectives for this project:

1. Perform analyses to determine required capacity of channels and inverts required to move flood waters and reduce flooding in Cashiers.
2. Analyze sediment loads and determine if there is a need for sedimentation traps within the area.
3. Review and recommend required State and Best Management Practices for stormwater and erosion control.
4. Determine maintenance intervals for channel sediment removal.
5. Investigate roadbed damage, estimate repair costs and frequency.
6. Identify and quantify use of Frank Allen Road; estimate economic impacts if roadway becomes unusable.
7. Prepare concept plans, rough order of magnitude time and cost estimates for design/construction of several alternatives.
8. Consider prioritizing features and sequencing construction to be adaptable to funding constraints.

This report is intended to provide officials in the City of Cashiers and Jackson County with a preliminary study addressing the flooding and sedimentation issues in Cashiers. The following study criteria have been developed through the collaboration between the design team and Jackson County's Planning Director.

The report is to include components to address the following:

1. Proposed solutions to the flooding and sedimentation issues that are effective on the immediate time scale.
2. Proposed solutions to the flooding and sedimentation issues that act on the long-term scale.
3. Evaluations of the proposed solutions addressing solution effectiveness as well as cost.

The economic evaluations will be used to drive community decision-making to begin addressing the stormwater issues. As a preliminary document, direction for any further investigations necessary for the implementation of the proposed solutions must be detailed.

4. Stormwater Engineering Design Approach Recommendations

Stormwater Conveyance and Sediment Load Analysis was performed with available data to determine siting and sizing parameters for detention basins. We reviewed the North Carolina Department of Environment and Natural Resources (NCDENR) Stormwater Best Management Practices (BMP) Manual and EPA guidance documents to ensure that the technical specifications of our recommendations are in compliance with federal and state requirements, including design, maintenance and inspection intervals, and costing.

4.1 Stormwater Conveyance Analysis

The proposed stormwater conveyance system will incorporate thorough measures to alleviate both the flooding and sedimentation problems. For this, it is important to understand the existing infrastructures and features within the flood impact zone and analyze each of their roles in the flooding and sedimentation issues.

Some of the existing components for conveyance of stormwater in the system are as following:

1) Streams
2) Culverts

4.1.1 Qualitative Analysis of Streams

Most of the water and sediments are transported by a network of streams. Cashiers Creek, as shown in Figure 12 below, accumulates the majority of the water from the drainage area. Each of the streams were inspected for their contribution towards flooding and sedimentation in the region, but only a few were analyzed for further study, based on their contribution to the flooding and sedimentation issues.

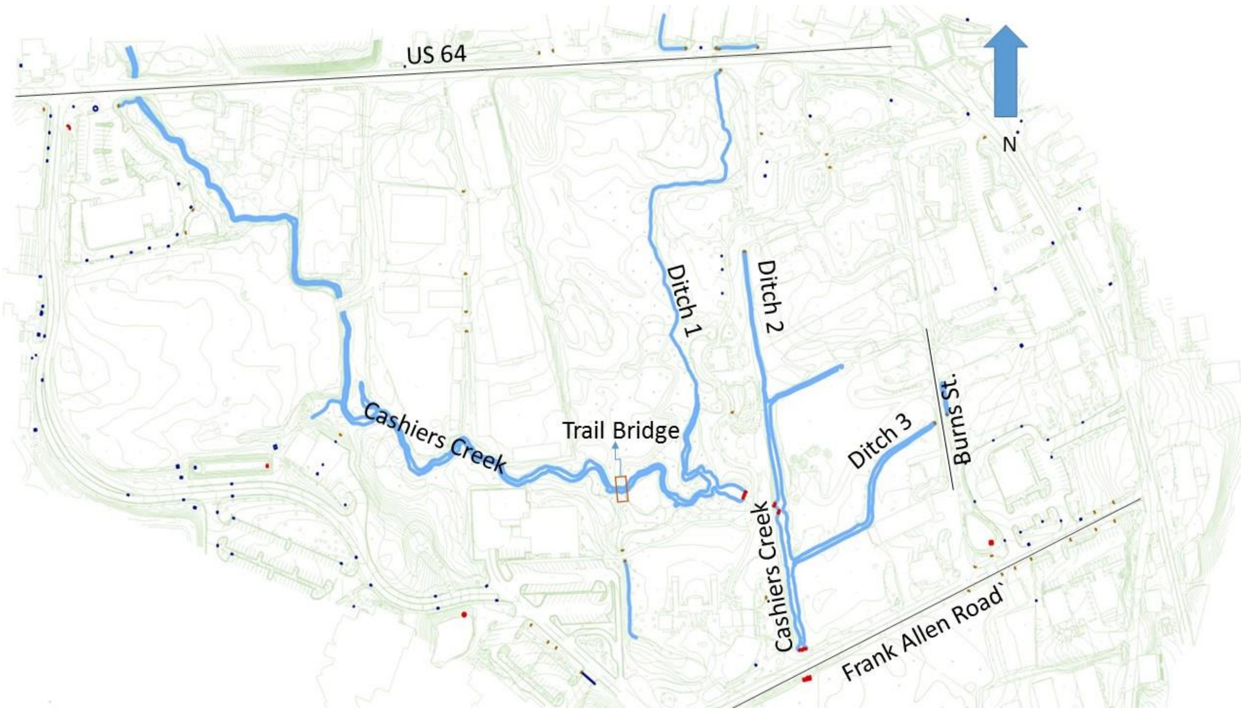

Figure 12. Network of streams between US-64 and Frank Allen Road

a) **Cashiers Creek:** Cashiers Creek flows through a high elevation region towards the entry point at US-64 and slows down rapidly. The drop in elevation before the entry point is around 20 feet over a stretch of 0.2 miles (1056 feet) which gives a slope of 0.019 ft./ft. whereas the drop in elevation after the entry point is about 6 feet over the same length which gives a slope of 0.0057 ft./ft. We can see that the region within the flood impact zone is not very steep. Figure 13 is a picture taken at the entry point of Cashiers Creek where sedimentation in one of the culvert is highly noticeable.

Figure 13. Sedimentation where Cashiers Creek enters under US-64

The sedimentation at the entry point is most likely due to the steep drop in elevation (approaching US-64) which causes the sediments to be transported rapidly and as the elevation gradient gets smaller (after the entry point), the flow slows down depositing large amount of sediments at that point.

Cashiers Creek, as it continues downstream, combines with Ditch 1, Ditch 2, and Ditch 3, carrying along with it a certain amount of sediment. Ditch 1, Ditch 2 and Ditch 3 also bring large amount of sediments as can be seen in the Figure 14 and Figure 15 below.

Figure 14. Ditch 3, along with its large amount of sediment deposition

Figure 15. Mouth of culvert connecting Ditch 1 and Ditch 2, after it branches from Ditch 1.

b) **Ditch 1:** This stream originates from the various runoff on the north east side of US-64. This stream also directs the water towards Cashiers Creek from various catch-basin and surface runoff on US-64. Figure 16 shows the starting point of Ditch 1 right beside the highway.

Figure 16. Ditch 1 by US-64 beside the road.

In the Figure 16, it is noticeable that the water carries a lot of sediments from the north of US-64, along with the sediments from the many gravel parking lots situated along the highway. This stream eventually branches off into Ditch 2 and the rest of it goes into Village Green Wetlands.

c) **Ditch 2:** As shown in Figure 15 above, the standing water is caused by the heavy sedimentation as well as the presence of vegetation in the stream. The sediments of this stream are not only a result of Ditch 1, but also the playground parking lot. Figure 17 is a picture of Ditch 2 runoff after a rainfall. The muddy water shows the sediment that is being carried by this stream; an extended analysis on sediment size distribution found at various parts of streams in section 4.1.2

Figure 17. Ditch 2, view is facing downstream towards Cashiers Creek

d) **Ditch 3:** This stream, as shown in Figure 12 above transports water from Burns St. and its surrounding, and directs the flow into Cashiers Creek. This stream also carries a lot of sediments into Cashiers Creek as shown in Figure 14 above. Most of the sediments look like they were remnants of what had been dug out to during the construction of this stream.

4.1.2 Qualitative Analysis of Culverts

Culverts are an important part of the stormwater conveyance system. Culverts are used in connecting streams under a road or a bridge, and the network of culverts found between US-64 and Frank Allen Road is show in Figure 18 below. The culverts shown below are responsible for conveying a certain discharge in the flood impact zone. From our analysis, it is observed that most of the culverts are clogged, which causes significant ponding in the region. A few of the important culverts will be discussed in this section to further our stormwater study of the affected region.

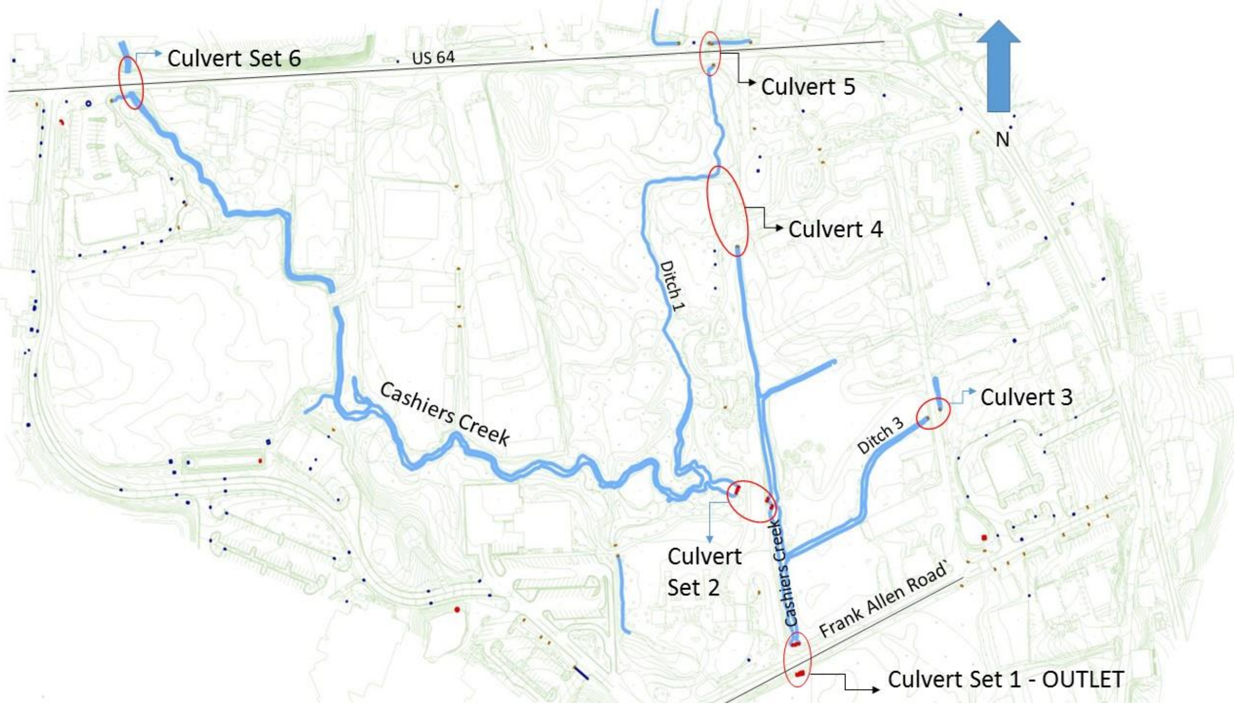

Figure 18. Culvert network flowing towards Frank Allen Rd.

a. **Culvert Set 6:** This is the first set of culverts which convey the discharge from north of US-64, along Cashiers Creek. As seen in Figure 13 above, one of the culvert is completely clogged with sediments. The other looks functional, but for peak discharge it will not be able to handle the flow causing more water to back up. These set of culverts are arched corrugated metal pipes (CMP) with dimension 81”×59”. These have a cross sectional area of 27.4 sq.-ft.

b. **Culvert 5:** The dimensions of this culvert are unknown due to visual obstructions, but it can be seen that it is used for directing water from the catch basins and other inlets north of US-64 and conveying it towards Ditch 1. This culvert has some amount of sedimentation, but not too much for it to cause water to back up significantly.

c. **Culvert 4:** This culvert is used at the branching of Ditch 1, which splits into Ditch 2 as shown in Figure 19 below. Again the dimensions of the culvert are not known, but sedimentation was observed in these culverts.

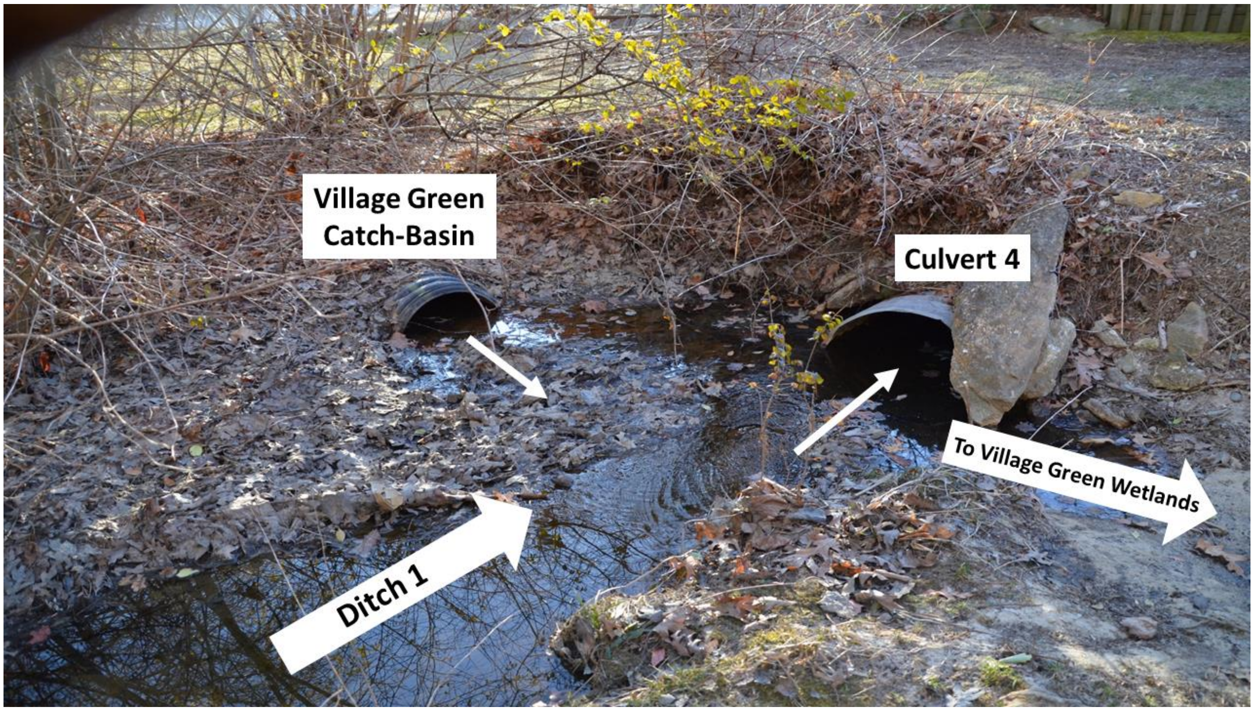

Figure 19. Ditch 1 intersection. This intersection receives flow from Ditch 1 and a catch-basin from a Village Green parking lot and conveys the flows to Culvert 4 and the Village Green Wetlands.

d. **Culvert 3:** Located to the east of Cashiers Creek, this culvert runs under Burns St in and connects water directed towards Ditch 3. This culvert has a high load of sediment, which can be seen in the Figure 20 below.


Figure 20. Inlet of culvert 3, with heavy sedimentation.

e. **Culvert Set 2:** These culverts are another set of Arch CMP with the same dimensions as Culvert 6. These culverts connect Cashiers Creek with Ditch 1. All of this flow is conveyed to a point north of Frank Allen Rd. A picture of Culvert Set 2 is shown in Figure 21 below.

Figure 21. Culvert Set 2 connecting east-side water into Cashiers Creek.

f. **Culvert Set 1:** These culverts serve as the outlet of our watershed. These culverts convey all the flow from the north of Frank Allen Road into Cashiers Lake Wetlands. These culverts have the same specification as that of Culvert 2 and 6 (Arch CMP 81”×59”) as shown in Figure 22 below.

Figure 22. Culvert Set 1 right below Frank Allen Rd.

4.1.3 Quantitative Analysis of Sediments

To understand the transport mechanism of these sediments it is important to analyze the grain size distribution of the sediments along with the velocity of the stream in order to determine the sizes of sediments that will erode, accrete or transport in the stream. These ranges can be shown in a Hjulström curve (as shown in Figure 23), which provides a basis for the design constraints.

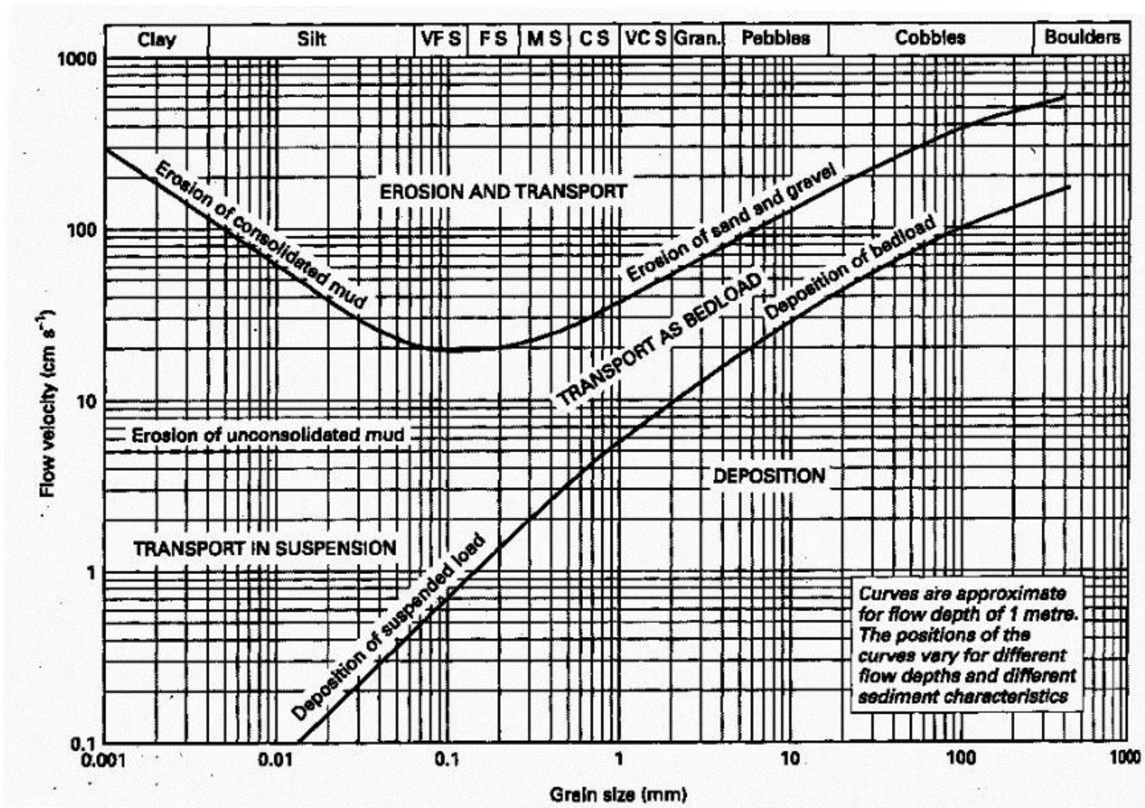

Figure 23. Channel flow velocity impact on various sized sediments shown by a Hjulström Curve. (Cool Geography 2015)

From Figure 23 it can be seen that a range of grain size and flow velocity will generate transport in suspension, as well as transport as bed-load. From the data generated through sieve analysis (Appendix D), we get the information tabulated in Table 3. Sieve analysis was also conducted for each sample using ASTM C136 Standard Test Method.

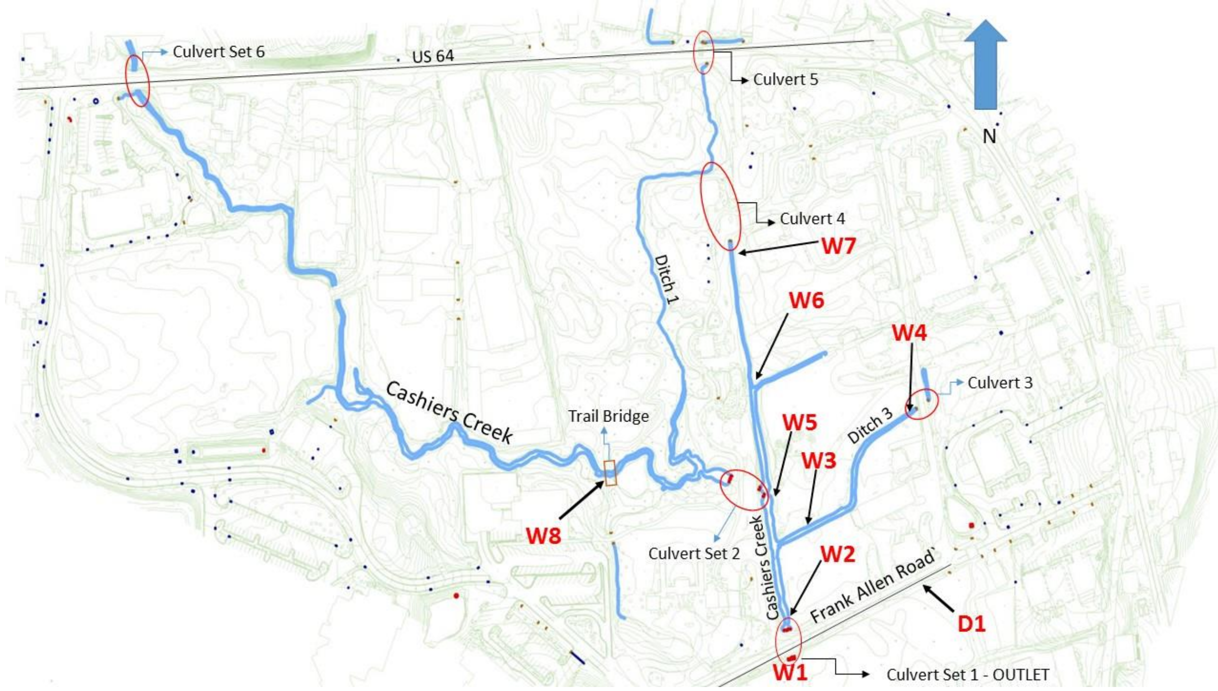

Figure 24. Locations of Sample Sediments.

Table 2. Sediment Sample Location Description.

Code	Location
W1	Downstream of Culvert Set 1. (Outlet of Frank Allen Culvert)
W2	Upstream of Culvert Set 1. (Inlet of Frank Allen Culvert)
W3	Ditch 3 (Right before it merges with Cashiers Creek)
W4	Downstream of Culvert 3
W5	Downstream of Culvert Set 2
W6	Along Ditch 2. ~500 ft. above Cashiers Creek and Ditch 3 Intersection.
W7	Downstream of Culvert 4, Along Ditch 2
W8	West Side Stream (Cashiers Creek) near the Trail Bridge
D1	Dry Sediment, picked up from Frank Allen Rd.

Table 3. Sediment size analysis for the median and mean particle size

Location	d_{50}(mm)	d_{84}(mm)	d_{16}(mm)	d_{avg}(mm)
W1	0.70	1.90	0.35	1.12
W2	0.61	1.40	0.40	0.90
W3	1.19	2.20	0.60	1.40
W4	3.00	8.00	0.80	4.40
W5	0.50	2.00	0.24	1.12
W6	0.34	0.53	0.11	0.32
W7	1.20	2.00	0.48	1.24
W8	0.61	1.50	0.35	0.93
D1	0.28	0.87	0.10	0.49

The west point of entry of Cashiers Creek at US-64 as shown in Figure 13 brings a lot of sediments from higher elevation region. The drop in elevation before the entry point is around 20 feet over a stretch of 0.2 miles (1056 feet) which gives a slope of 0.019 ft./ft. whereas the drop in elevation after the entry point is about 6 feet over the same length which gives a slope of 0.0057 ft./ft. This rapid change in slope causes most of the sediments to deposit on the entry point by US-64. As Cashiers Creek flows downstream a lot of the heavier sediments are left behind. A sample collected near the Trail Bridge on Cashiers Creek had an average particle diameter of 0.93 mm.

W6 is much smaller than W7 although they are on the same stream. This might be the cause of sediments settling on the downstream of Culvert 4. The sediment size nevertheless increases around the outlet of Culvert Set 2, as shown by W5. This is the place where the streams from the west-side converge into the east-side stream, transporting larger sediments into Cashiers Creek.

Tracing the sediments down south, we see that the sediment size on the inlet of Culvert Set 1 decreases in size, but not significantly, when compared to W5. W2 is around 0.90 mm whereas W5 is around 1.12 mm. Although there is some settlement around Culvert Set 2, a lot of it gets clogged right at the inlet. This can again be verified by the increase in sediment size downstream of Culvert Set 1 (at the outlet) where sample W1 has an average particle diameter of 1.12 mm. This is contrary to the flow behavior; therefore we can say that the sediments are not getting transported properly, and this could be the result of erosion or deposition within the stream due to its velocity.

4.1.4 Quantitative Analysis of Stormwater

Stormwater analysis is performed in accordance to the North Carolina Stormwater Best Management Practices (BMP) Manual.

1) **Peak Flow Measurement:** The peak flow must be attenuated such that a 2-year, 24-hour post-development storm wouldn't exceed the pre-development flow rate (NC Stormwater BMP Manual 2007). The information on the watershed is given in a report by LandDesign (Appendix C). The data given in the report were gathered by the crew in 2012 during the Cashiers Dam remediation project.

   The dam report has information on the peak flow and the time concentration of the watershed defined with the outlet located at Cashiers Lake as shown at the lower part of Segment #5 in Figure 25. Since we are only concerned with the flow up to Frank Allen Rd. (Culvert Set 1) we can exclude the extra length (From Frank Allen to the lower part of the Segment #5) and modify the parameters based on the new length. The data for all the segments are provided below.

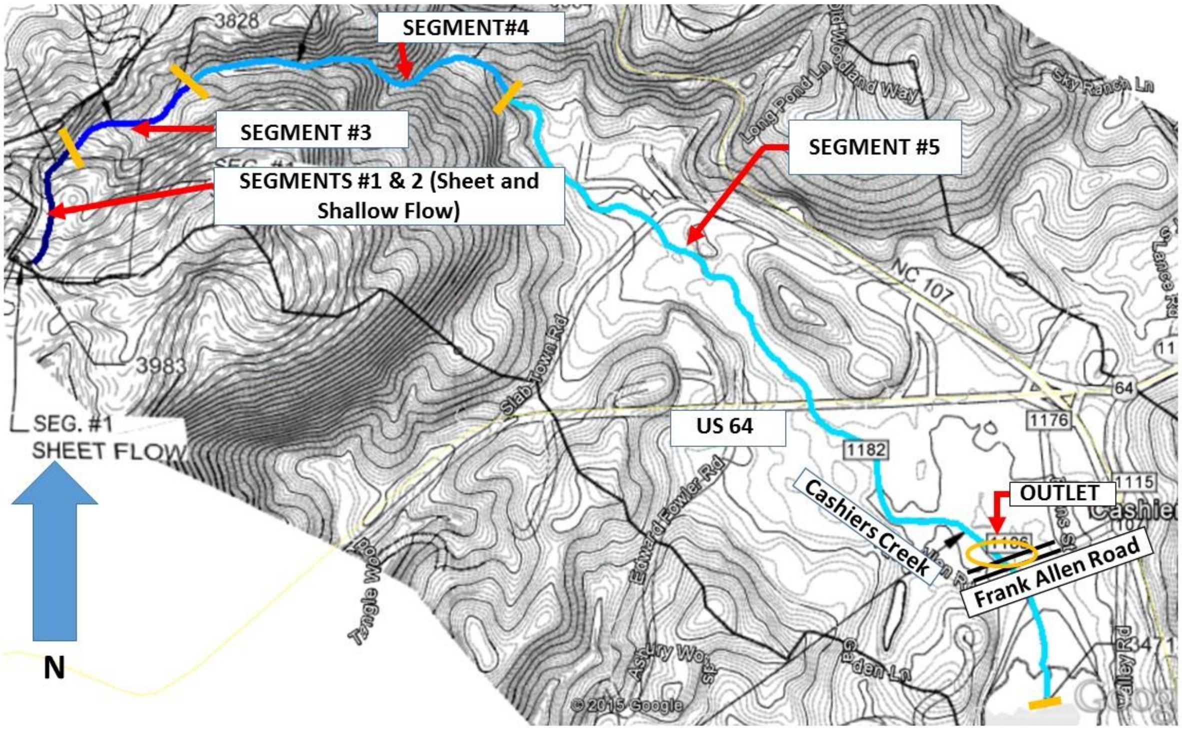

Figure 25. Drainage Area of Culverts Set 1 (Frank Allen Rd culverts).

The information as given in the dam report (Appendix C) for each stream segment, along with modified segment #5 are tabulated below.

Table 4. Segment #1 Sheet Flow

Mannings, n	0.400
Hydraulic Length (ft.)	100
2 yr, 24 hr., Precipitation, P (in.)	5.71
Slope (ft/ft)	0.02
Avg. Velocity	0.10
Time concentration, Tc (hr.)	0.2679
Peak Flow, Q (cfs)	N/A

Table 5. Segment #1 Shallow Flow

Hydraulic Length (ft.)	707
Slope (ft/ft)	0.105
Avg. Velocity	5.23
Time concentration, Tc (hr.)	0.0376
Peak Flow, Q (cfs)	N/A

Table 6. Segment #3 Channel Flow

Flow Area, A (sq.-ft.)	5.0
Hydraulic Length (ft.)	903
Wetted Perimeter (ft.)	8.33
Hydraulic Radius (ft.)	0.60
Mannings, n	0.048
Slope (ft./ft)	0.0906
Avg. Velocity	6.65
Time concentration, Tc (hr.)	0.0377
Peak Flow, Q (cfs)	33.25

Table 7. Segment #4 Channel Flow

Flow Area, A (sq.-ft.)	6.0
Hydraulic Length (ft.)	2296
Wetted Perimeter (ft.)	9.33
Hydraulic Radius (ft.)	0.64
Mannings, n	0.048
Slope (ft./ft.)	0.115
Avg. Velocity	7.85
Time concentration, Tc (hr.)	0.0813
Peak Flow, Q (cfs)	47.1

Table 8. Segment #5 (Modified) Channel Flow (Appendix E)

Flow Area, A (sq.-ft.)	20
Hydraulic Length (ft.)	5500
Wetted Perimeter (ft.)	16.65
Hydraulic Radius (ft.)	1.20
Mannings, n	0.060
Slope (ft./ft.)	0.014
Avg. Velocity	3.50
Time concentration, Tc (hr.)	0.36
Peak Flow, Q (cfs)	56.20

The total time concentration, Tc = 0.78 hr.

Peak flow-rate, Q = 56.20 cfs.

Remarks: We can see that the peak flow rate increases as we go downstream gradually. The peak flow-rate calculated in this part is the post-development scenario. The pre-development scenario discharge is approximately 33 cfs.

The velocity of the channel is 3.50 ft/s which corresponds to 107 cm/s. From Fig 23, transport is possible for very small grain sizes, and evidently most of the sediments are getting eroded as a result of this high velocity, and as the sediments approach Frank Allen Rd. the significant ponding causes most of the sediments to settle. For this, we need to improve conveyance of the flow through the Culvert Set 1 into Cashiers Lake Wetlands. In the next sections, we will review state BMP and analyze the infrastructures to make recommendations to mediate this issue.

4.2 Review of Best Management Practices

In order to develop effective stormwater recommendations, Renovatus Solutions conducted a review of North Carolina's State Best Management Practices (BMPs) for water pollution. Because the state BMP manual contains technologies and specifications recommended by the Division of Water Quality that meet minimum regulatory stormwater requirements set by the state, we evaluated the state BMP manual to be an appropriate source for BMPs.

4.2.1 Structural BMPs

Effective BMP selection entail many variables. Criteria for selection usually falls under three main categories: BMP treatment capabilities, BMP site constraints, and BMP cost and community acceptance. Because pollutant removal is not a priority in this study, selection only due to siting constraints, cost, and community acceptance were considered.

Table 9 is taken from the North Carolina Department of Environment and Natural Resources NCDENR BMP Manual and outlines BMP site constraints based on different factors. Based on our site assessment of Cashiers, we determined that the factors with the most weight were the size of drainage area, workability with shallow water table, workability with high sediment input, and workability with poorly drained soils.

Table 9. NCDENR BMP Site Constraints (NCDENR 2012).

BMP	Size of Drainage Area	Space Required	Head Required	Works with Steep Slopes?	Works with Shallow Water Table?	Works with Shallow Depth to Bedrock?	Works with High Sediment Input?	Works with Poorly Drained Soils?
Bioretention without IWS	S	High	Med	Y	N	N	N	Y
Bioretention with IWS	S	High	Med	Y	N	N	N	N
Stormwater wetlands	S-L	High	Med	N	Y	N	Y	Y
Wet detention basin	M-L	High	High	N	Y	N	Y	Y
Sand filter	S	Low	Med	Y	N	N	N	Y
Filter strip	S	Med	Low	N	Y	Y	N	Y
Grassed swale	S	Low	Med	Y	Y	N	N	Y
Restored riparian buffer	S-M	Med	Low	N	Y	Y	N	Y
Infiltration devices	S-M	High	Low	N	N	N	N	N
Dry extended detention basin	S-L	Med	High	N	N	N	Y	Y
Permeable pavement system	S-M	N/A	Low	N	N	N	N	Y
Rooftop runoff management	S	Variable	Low	Y	Y	Y	Y	Y

As mentioned in previous sections, the drainage area of the BMP was estimated to be 480 acres, which would make classify it as a medium sized drainage area. From site visits and reports, the flooded area of Cashiers was determined to have a shallow water table. Therefore, BMPs that are constrained by workability with shallow water table cannot be effectively implemented unless they are placed farther from the flooded area, where the water table is deeper. Because a high sediment input was observed in the existing stormwater system, BMPs that are not able to handle significant sediment loads were excluded from the criteria selection. Finally, poorly drained Nikwasi soil was found to be prevalent in Cashiers, especially in the flooded area. BMPs that mainly rely on infiltration would not be effective and were not considered.

Based on the factors discussed above, BMPs were eliminated to arrive at the following potential candidates: stormwater wetland, wet detention basin, and dry extended detention basin.

4.2.1.1 Stormwater Wetland

Stormwater wetlands are designed and constructed to mimic the functions of natural wetlands. They have the capability to drain small to large drainage areas. Areas with a shallow water table present no issues because by design stormwater wetlands are meant to contain a permanent pool of water. They are overall best BMPs for pollutant removal and can have up to 85% removal efficiency of total suspended solids, including sediments. Figure 26 shows an example of a stormwater wetland. Recommended and required design elements by NCDENR can be seen in Table F-1 (Appendix F).

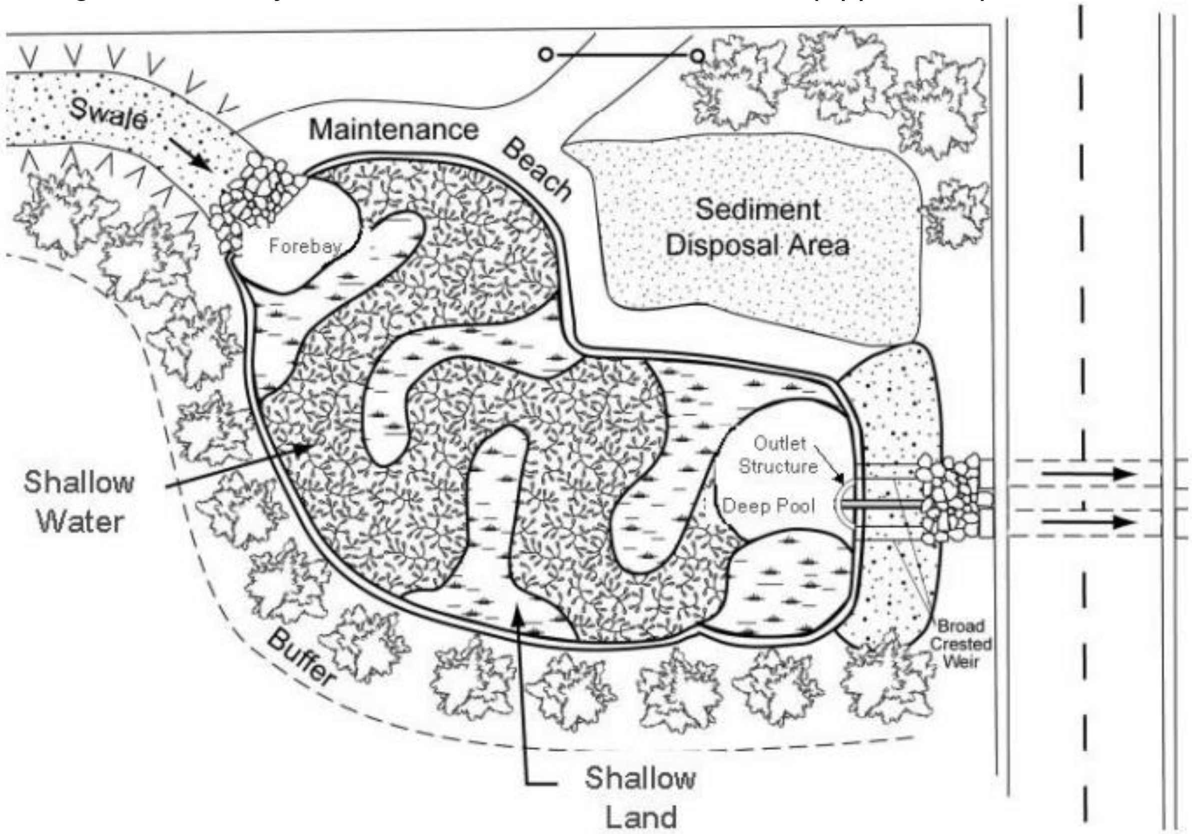

Figure 26. Sample Stormwater Wetland Plan View (NCDENR 2009a).

4.2.1.2 Wet Extended Detention Basin

Wet detention basins are structures that contain a permanent pool of water for removing sediment and pollutants from stormwater. In addition to its permanent capacity, a wet detention basin can have additional volume for detaining stormwater runoff. They are ideal in draining medium to large drainage basin sizes as they have the runoff volume required to maintain a permanent pool of water. Because they are required to have a permanent pool of water by design, wet detention basins are appropriate for areas that have a shallow water table and are very efficient at removing suspended solids from incoming stormwater. They are compatible with locations containing poorly drained soils, as they do not rely chiefly on infiltration to convey detained runoff. Figure 27 shows a typical wet extended detention basin and its features. Recommended and required design elements by NCDENR can be seen in Table F-2 (Appendix F).

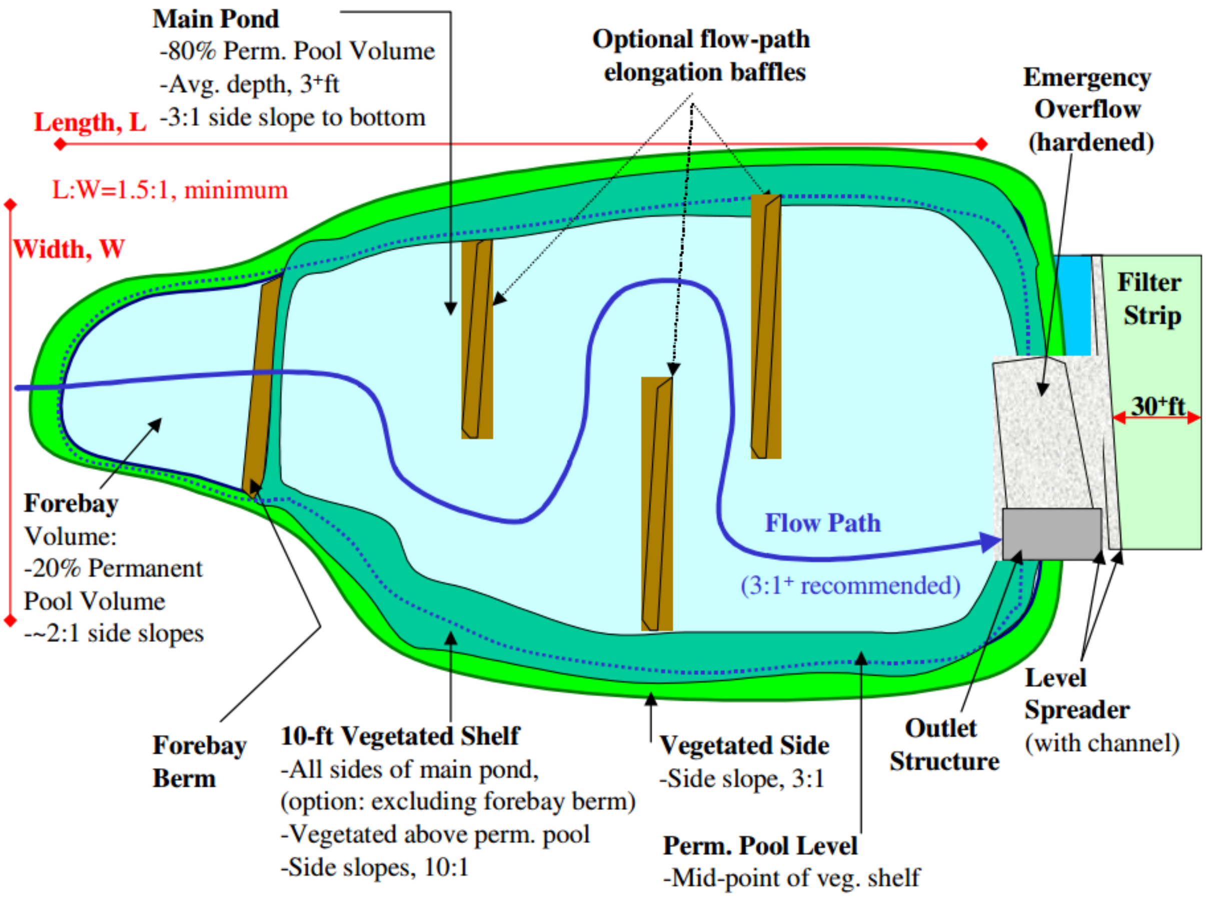

Figure 27. Wet Extended Detention Basin (NCDENFR 2009b).

4.2.1.3 Dry Extended Detention Basin

Dry extended detention basins temporarily store stormwater runoff to reduce peak discharge and are dry in between storm events. These basins can offer much more effective peak attenuation for small and large drainage areas, but because they do not have a permanent pool of water, they are not as successful in removing suspended solids, having a removal efficiency of about 50%. Because they must dry out in between storm events, their viable locations are limited to only areas with a deep water table. Like the previously mentioned BMPs, dry detention basins rely on the slow and controlled release of detained water rather than infiltration, making them suitable for areas with poorly drained soils. Figure 28 shows a diagram of a typical dry detention basin and its features. Shows a typical wet extended detention basin and its features. Recommended and required design elements by NCDENR can be seen in Table F-3 (Appendix F).

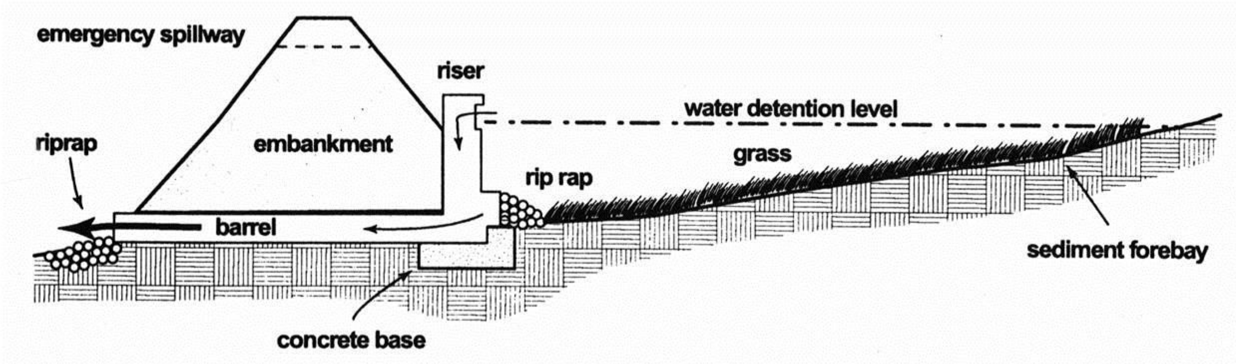

Figure 28. Sample Dry Detention Basin Plan View (NCDENR 2009c).

4.2.2 Nonstructural BMPs

In this project, nonstructural BMPs apply to sediment removal from Cashiers Creek and the wetland area and to maintaining stormwater culverts. According to the 2007 NCDENR BMP manual, removal intervals for accumulated sediment vary dramatically among facilities and depend on rate of sedimentation. Sediment removal from the stream and culverts should occur regularly; however, the frequency of this maintenance needs to be determined. We recommend sediment removal from Cashiers Creek and the wetland area every four years. This time period can be adjusted based on need. We recommend inspecting stormwater culverts every year for damage and sedimentation. A maintenance schedule can be created based on observations from these inspections.

Disposal of removed sediment is required. If the disposal site is located nearby, it must be outside of the floodplain (NCDENR 2007). Transportation and landfill tipping fees can increase the cost greatly, and sediment may need to be set aside to dry before being sent to a landfill (NCDENR 2007).

4.3 Stormwater Infrastructure Recommendations

Based on the findings in stormwater conveyance analysis and selection guidance in the BMP review, Renovatus Solutions recommends two complementary solutions: 1) stormwater detention and sediment capture and 2) sediment removal and wetland dredging. By alleviating the high sediment load and temporarily storing incoming excess stormwater, the solutions will work in tandem to reduce the flooding caused by the flat topography and poorly drained Nikwasi soil.

4.3.1 Stormwater Detention and Sediment Capture

There are many design aspects which should be considered when designing a detention basin specifically for a location. Some of the distinct features among which we must make our decision are shown below:

1) **Underground vs. Aboveground:** The underground structure is only necessary if the overground surface area is insufficient to hold the volume of water required for a storm event (Haestad and Durrans 2003).

2) **On-Line vs. Off-Line:** These are the various pathways a stream can take in order to get to the detention pond. An on-line pond is located along the pathway of the runoff whereas the off-line pond requires the runoff be diverted into the pond. For the purposes of our design, we will be using an on-line detention basin such that the diversion of the stream is not required (Haestad and Durrans 2003).

To determine the required storage volume for the detention facility, we use parameters for pre- and post- development peak flow and its respective time concentration. Time of concentration is the amount of time it takes water to flow from the farthest part of the watershed to a defined point. Peak flow and time concentration parameters help determine the storage volume of a detention basin.

To limit the peak discharge, we need post-development discharge to be smaller than that of pre-development (Haestad and Durrans 2003). It is important to mimic the pre-developed conditions as they simulate areas with fewer impervious surfaces. The time it takes for water to flow from one point to another is shorter on impervious surfaces, therefore this decrease in the time concentration increases the flowrate. To attenuate this flowrate, a certain amount of water must be detained, and this volume is determined using the difference in the peak runoff rate for the two conditions.

As the discharge increases in the streams, Culvert Set 1 will need to convey more water into the Cashiers Lake Wetland, which has the poorly drained Nikwasi soil. This poorly drained soil retains large volumes of water causing Cashiers Creek to back up onto Frank Allen Road. This ponding then causes sedimentation of sand grains, which clogs the streams and culverts even more.

For effective detention of the stormwater runoff we will perform two different analyses. One with a single basin located at site shown in Figure 29, and another as a combination of the two sites shown in Figure 31. With the two options, we will analyze the effectiveness of the basin for sedimentation and flood control.

Table 10. Cashiers Precipitation for a 2-yr Recurrence Interval (Appendix G)

Storm	24-Hr. Rainfall Depth, in.
2 year	5.92
5 year	7.28
10 year	8.38

Table 11. Pre- and Post-Development Watershed Conditions (Calculations shown in Appendix H)

Watershed conditions	Area (ac.)	Time Concentration,Tc (hr.)	Flowrate (cfs)
Predevelopment	480	0.91	33
Post development	480	0.78	56

Using these parameters, the detention basin must be sized at 1.13 ac-ft., or approximately 50,000 cu.-ft.

4.3.1.1 Single Basin Option

The Single Detention Basin Option is composed of a single wet extended detention basin near the outlet of the stormwater system (See Figure 29)

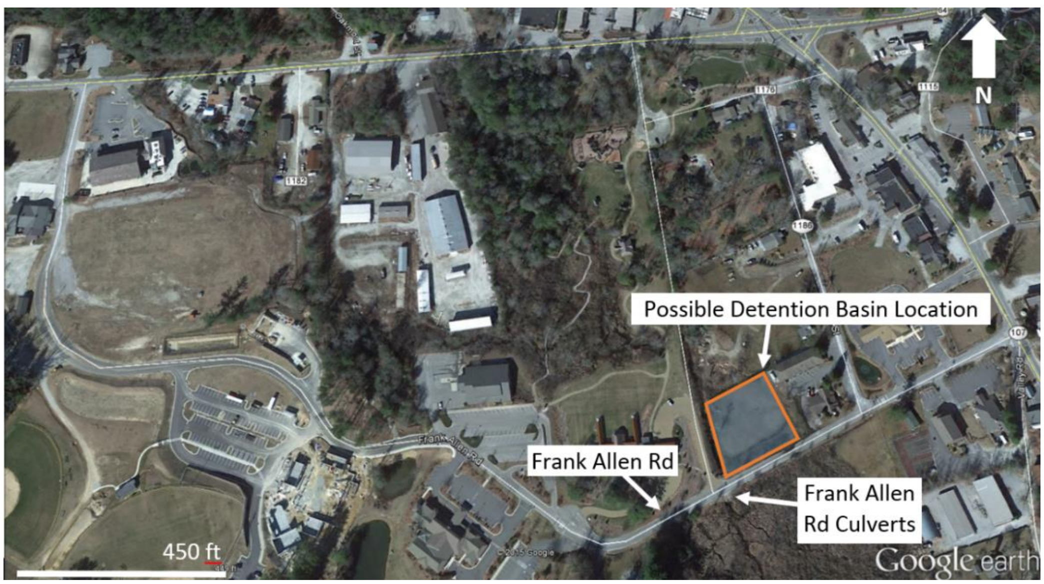

Figure 29. Single Detention Basin Option

Placing a single basin near the outlet of the stormwater system produces the simplest solution. Less variables need to be considered and the calculations required are less complex because there are very few downstream effects to account for. The site of single detention basin is also suitable to trap sediments directly before they settle in front of Culvert Set 1. This is important as the sediments that settle in this location can clog the culverts.

The constraints of the location, however, limit the selection of BMPs to a wet extended detention basin or a stormwater wetland. Because pollutants are of little concern and because it is less expensive to construct and maintain, we recommend the use of a wet extended detention basin.

From the stormwater conveyance analysis, we have determined that the single basin option will necessitate a minimum detention volume of approximately 50,000 cubic feet. The use of baffles and retaining walls are recommended to maximize the volume of the main pond given the area constraints. Connecting the outlet structure to the Frank Allen Rd culverts is recommended. A rough design of the wet detention basin can be seen in Figure 30.

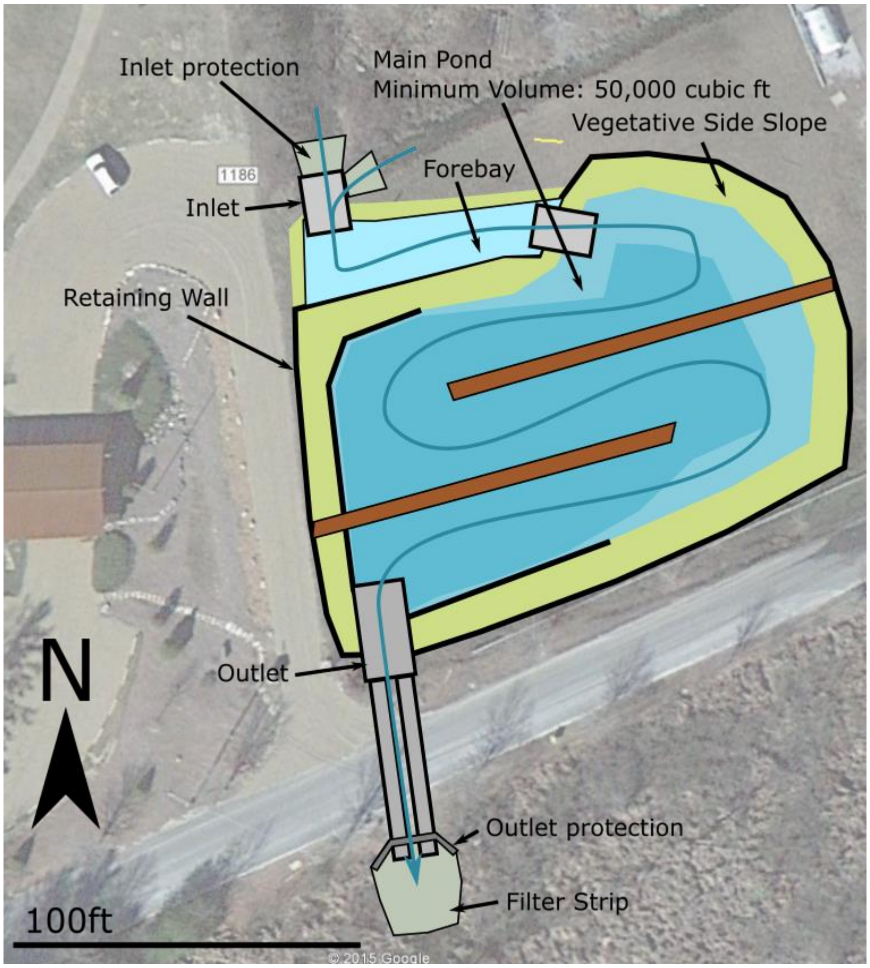

Figure 30. Wet detention basin design for the single basin option.

4.3.1.2 Double Basin Option

The Double Basin Option builds upon the Single Basin Option by adding a dry extended detention basin further upstream of Cashiers Creek (see Figure 31).

Figure 31. Double Detention Basin Option

The upstream location has been shown to have a deeper water table and allows for the use of dry detention basins, wet detention basins, and stormwater wetlands. The combination of the two will detain more water reducing the overall outflow discharge significantly. This option will also trap sediments effectively at two places reducing the overall load. The storage volume depending on the pre and post development parameters are tabulated in Table 12 and 13 below for the two different sites.

Table 12. Site next to Frank Allen Road. (Calculations shown in Appendix H)

Watershed conditions	Area (ac.)	Time concentration,Tc (hr.)	Flowrate (cfs)
Predevelopment	480	0.91	18
Post development	480	0.72	33
Storage Volume	0.78 ac-ft. / 34,000 cu-ft.		

Table 13. Site next to US-64 (Calculations shown in Appendix H)

Watershed conditions	Area (ac.)	Time concentrationTc (hr.)	Flowrate (cfs)
Predevelopment	480	0.62	33
Post development	480	0.72	60
Storage Volume	0.787 ac-ft. / 34,000 cu-ft.		

4.3.1.3 Comparison of Options

Table 14. Single vs double basin comparison

Parameter	Single Basin Option	Double Basin Option
Detention Effectiveness		X
Sediment Removal		X
Cost	X	
Feasibility	X	
Environmental impacts	X	

Table 14 shows the pros and cons of the two options. This comparison will be the driving factor for decision making and further evaluation in Section 4.6.1.

4.3.1.4 Environmental Impacts

The construction and presence of detention basins in a watershed can have the potential to severely disrupt pre-existing conditions. Environmental laws, such as Section 404 of the Clean Water Act (CWA) have been put in place by the US government to protect jurisdictional waters of the US and are enforced via regulatory permits by the USACE.

To reduce the environmental impact of the detention basins, appropriate BMPs and design parameters must be employed. To mitigate unwanted sediment transport during construction, BMPs such as silt fences can be utilized. Sustainable reuse of dredged material during construction can be employed (see Section 4.3.3.1). Design peak discharge rate of the basin can be controlled such that it does not exceed preexisting conditions to avoid any potential disruption downstream.

It has been determined by the Wilmington District Corps of Engineers that there are currently jurisdictional streams and wetlands in the proposed work areas. Before moving further to any site work or preparation of final design plans, it is highly recommended by the Corps to perform a jurisdictional determination of the area to define any jurisdictional features and their limits.

4.3.2 Sediment Removal

To alleviate flow to Cashiers Lake, lower stream levels, and lower the water table in the area, we recommend removing sediment from Cashiers Creek north and south of the culverts on Frank Allen Road. The recommended sediment removal area is shown below in Figure 32.

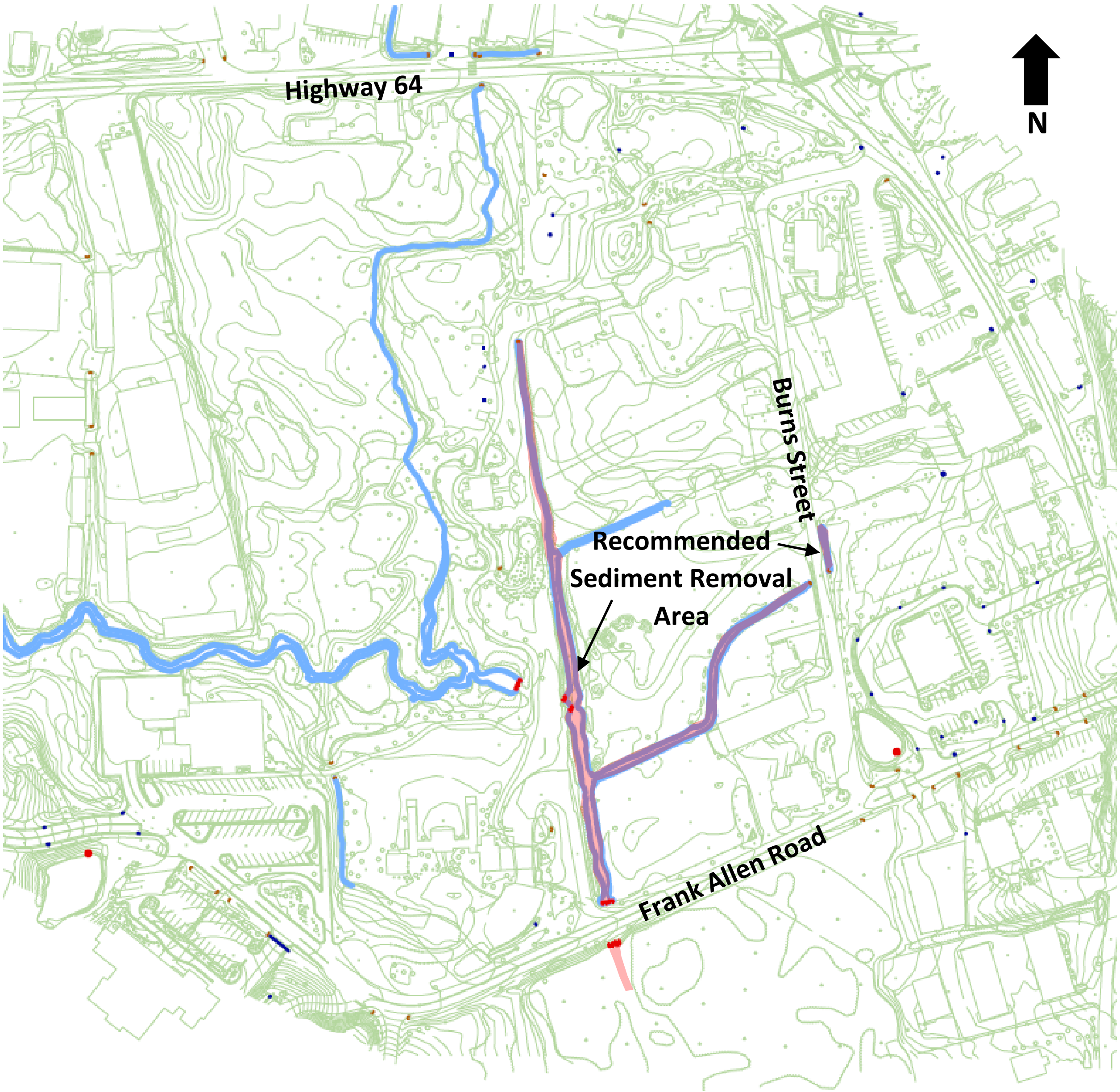

Figure 32. Recommended Sediment Removal Area

Sediment removal from the creek can be accomplished using a Spyder hoe excavator to minimize impacts to the environment and use in high water. A Spyder hoe is an excavator mounted on four articulated walking legs, allowing the machine to stabilize itself on uneven terrain and operate in over five feet of water. The machine is lightweight compared to other back hoes and includes biodegradable hydraulic fluid to reduce environmental impacts. Approximately six inches in depth of sediment should be removed from the creek to restore the stream bed to its previous elevation of 3472.9 feet. This elevation corresponds to that of the culvert inlets north of Frank Allen Road. Sediment removal should also occur south of Frank Allen Road. However, the creek was not mapped there. Further survey will be required to determine how far downstream the creek can be dredged. Measures should be taken during the sediment removal process to prevent further sedimentation downstream of the project area. This can be achieved by installing a sediment barrier before sediment removal begins. An example of sediment barrier placement is shown below in Figure 33. Sediment removal should be scheduled to take place in the winter months (November to March) to avoid elevated water levels from summer storms and increased traffic from tourists and seasonal residents.

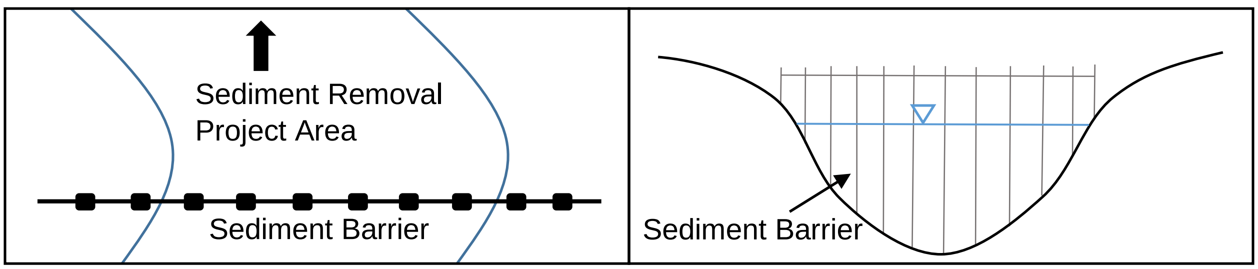

Figure 33. Sediment Barrier Schematic, Overhead and Cross-Sectional View

The creek is fairly shallow (approximately 5 to 6 feet deep), so sediment removal with a Spyder hoe should be reasonable without the need to divert the flow around the project area. However, if diversion of the creek is required, the most feasible option is pumping due to lack of space around the creek. This method would involve blocking the creek upstream and downstream of the project area and pumping the flow around. Diverting the flow will cause the cost of dredging to increase.

After the sediments has been removed, it can be trucked offsite to be disposed of or reused. See section 4.3.3.1 for details about sediment reuse options. Possible environmental impacts of sediment removal are also located in section 4.3.3.1. The estimated capital cost of Sediment removal is $22,000. More information about what is included in this estimate and how it was calculated is discussed in section 4.4.3.

4.3.3 Wetland Dredging

In addition to removing sediment from Cashiers Creek, we recommend dredging the wetland area leading into Cashiers Lake. As shown in Figure 34, the stream can be seen cutting through the wetland. It appears that much of the stream is narrow and overgrown with vegetation. Additional surveying is required to determine the extent of blockage due to sedimentation and vegetation.

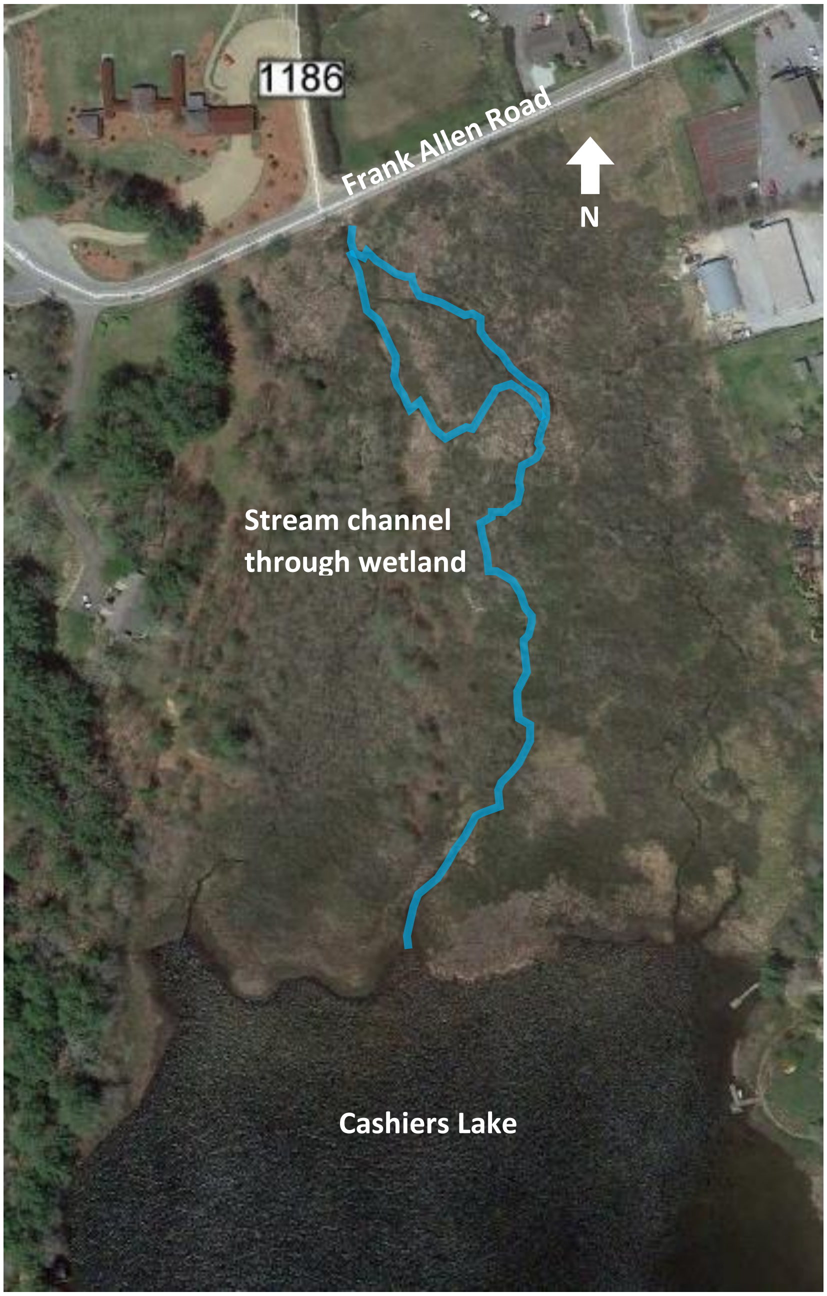

Figure 34. Stream Channel Through Cashiers Lake Wetlands

Dredging of the wetland can occur in conjunction with sediment removal from the stream, or the process can be delayed as sediment removal from the stream alone may be sufficient. If dredging of the wetland does occur, sediment and vegetation should be removed to restore the natural stream channel to Cashiers Lake. This increase in stream cross-sectional area will improve the conveyance of stormwater through the channel and reduce flooding. The improved conveyance will ensure that water level does not back up through the creek and overtop Frank Allen Road. Wetland dredging should take place during the winter (from November to March) to avoid high water levels associated with summer storms. Dredging during the winter will also ensure minimal impact to any wildlife in the area.

In addition to dredging the wetland, we recommend installing walking trails through the area. These trails can serve to facilitate continued monitoring and maintenance of the wetland after dredging, as well as provide a recreational nature walk for locals and tourists. Design and cost estimation of a trail through the wetland will require an in-depth survey of the wetland area.

It is estimated that the cost of dredging the wetland is $50,000. More information about what is included in this estimate and how it was calculated is located in section 4.4.

4.3.3.1 Environmental Impacts & Disposal

Sediment removal and disposal may cause unfavorable impacts to any animal and plant life in the creek. From our observations during the site visit, it appears that Cashiers Creek is not a fish bearing stream; however, further investigation is required to confirm this. Possible impacts include:

- Removal of subtidal benthic species
- Resettling of sediment over benthic species
- Short-term increase in turbidity which can effect animal and plant species
- Release of organic or nutrient rich sediments
- Release of contaminated sediments

Benthic animals (those that live on and in the sediment) are almost completely removed during dredging events. Studies have shown, however, that this impact is short-term and recovery of these species can occur between two months to a year after dredging (UK Marine SAC 2001). Resettling of sediments on top of benthic species may cause smothering leading to reduced rates of growth or, in the worst case, death of these species. Some species are more susceptible to smothering than others (UK Marine SAC 2001).

The increase of suspended sediments and turbidity from dredging and disposal under certain conditions can have adverse effects on marine species. Suspended solids reduce light penetration in the water column and produce physical disturbance. This can affect filter feeding organisms through clogging and damaging feeding and breathing organs (UK Marine SAC 2001). Young fish are affected similarly with the clogging of their gills, but adult fish are more likely to move away from turbid areas (UK Marine SAC 2001). Submerged plants and seaweeds are affected by the loss of light. Generally, the effects of increased turbidity due to dredging are short term (less than one week) and there should only be concern if sensitive species are in the area (UK Marine SAC 2001).

If the released sediment is rich in organic matter, depletion of oxygen in the water can occur. This depletion is temporary and oxygen levels will return to normal quickly as water flows through the channel (UK Marine SAC 2001). The only time this oxygen depletion is of concern is during important life stages of sensitive species (UK Marine

SAC 2001). The possible increase in nutrients can lead to eutrophication in some conditions, especially during the spring and summer. In other cases, increased nutrients may provide food for zooplankton and other organisms leading to beneficial environmental effects (UK Marine SAC 2001).

It is possible that the sediments are contaminated with substances such as heavy metals, oils, and pesticides. The sediment removal and disposal process may release these contaminants into the water column (UK Marine SAC 2001). These contaminants can then be taken in by plants and animals causing either contamination or poisoning. The likelihood of this occurring depends on the amount of contamination in the sediment. Low levels of contaminates are not a concern (UK Marine SAC 2001).

Because Cashiers is not located near the coast, disposal of the sediments in the ocean is not a feasible option. The most sustainable alternative is reuse of the sediment. There are several options for beneficial reuse of the removed sediment that should be considered. These include use as fill material for new land areas within ports and harbors, as landfill caps and covers, in road construction, for beach nourishment, or for topsoil creation and enhancement (University of Wisconsin Sea Grant Institute 2013).

4.3.4 Culvert Cleaning

Based on the analysis of the flood impact area, we have concluded that many of the culverts require cleaning. Cleaning the culverts will remove blockage resulting from sedimentation and vegetation. Removing the blockage will allow for increased capacity and better flow through the system. From visual inspection during the site visit, we have determined that the following culverts require maintenance: Culvert Sets 1 and 2, Culverts 3 and 4, and Culvert Set 6. The locations of these culverts are shown below in Figure 35.

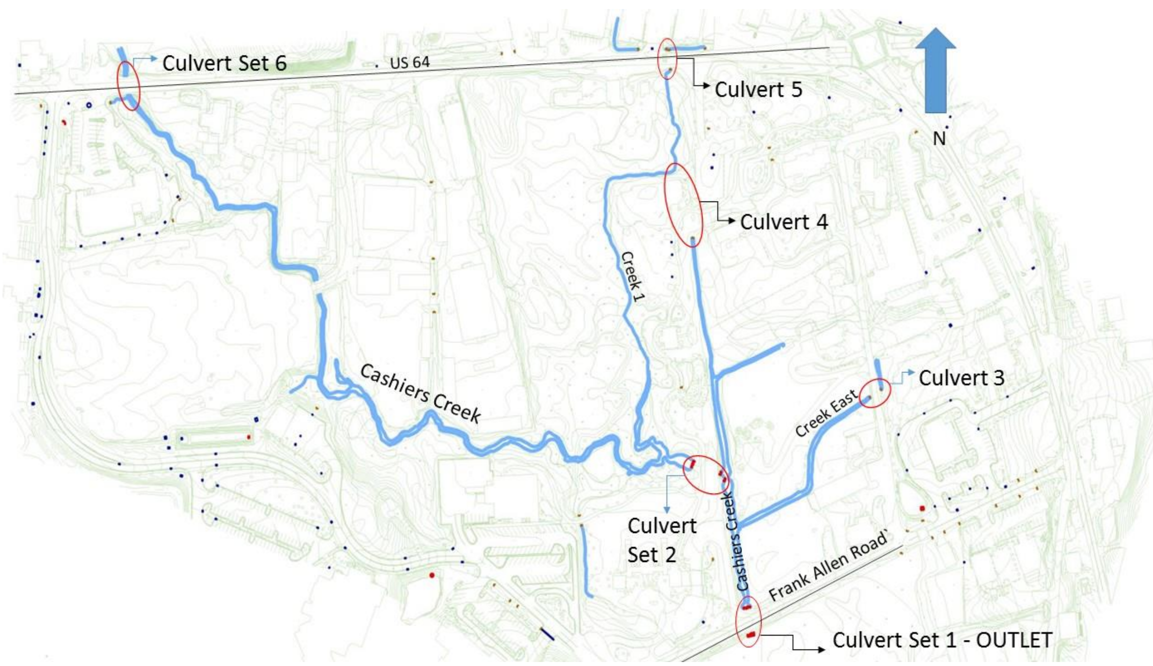

Figure 35. Culvert Locations

The North Carolina Department of Transportation is responsible for the maintenance of the culverts in the area, so their cooperation is required for this phase of the project. The larger culverts across Frank Allen Road and upstream along Cashiers Creek should be cleaned during, right before, or right after the sediment removal process. If the culvert cleaning is done in tandem with sediment removal, all of the debris and sediments can be disposed of at the same time. For the larger culverts, sediment and vegetation can be removed with shovels. Smaller culverts require a vacuum pump or cables in addition to shovels.

For a cost estimate of culvert maintenance, see section 4.4.

4.4 Cost Estimations

The following cost estimates are preliminary and do not represent a final cost evaluation. Costs are presented in the form of present costs and annual costs. All costs have an error margin of ±50%.

4.4.1 Single Basin Option

The construction costs for detention basins vary widely, mainly depending on the design volume. USEPA estimates base construction costs of wet detention basins and dry detention basins with adapted equations from Brown and Schueler, 1997 (Table 15).

Table 15. Detention Basin Cost Estimation Equations (USEPA 1999). Equations were adapted from Brown and Schueler, 1997.

BMP Type	Cost Equation
Dry Detention Basin	$C = 7.47V^{0.78}$
Wet Detention Basin	$C = 18.5V^{0.70}$

Where
C = Base construction cost
V = Volume needed to control the storm, in cubic feet

Based on the equation above, the base cost of wet detention basin with a volume of 50,000 cubic feet was calculated. Contractor, design, and legal/administrative fees were subsequently calculated from percent estimates of base costs. A 30% contingency was allocated for engineering detail and annual operational and maintenance (O&M) costs were set as 5% of the total capital cost. Present cost and annual cost were calculated using engineering economics formulas. A breakdown of individual cost items can be seen in Table 16.

Table 16. Cost breakdown for single detention basin option. Present and Annual costs were calculated based on an inflation rate of 3% and a lifetime of 20 years.

Cost Item	Percent of Subtotal (%)	Cost
Base Cost		$36,012
Contractor OH & Profit	15	$5,402
Contractor MB&I	5	$1,801
Engineering	25	$9,003
Legal, Administrative	15	$5,402
Subtotal (constructed cost, incl. contractor, design, and legal/admin. Fees)		$57,620
Contingency	30	$17,286
Total Capital Cost		$74,906
Annual O&M		$3,745
Present Cost		$130,000
Annual Cost ($/year)		$8,800

4.4.2 Double Basin Option

Using the same equations in Table 15, the base costs of a wet detention basin and a dry detention basin both with volumes of 34,000 cubic feet were calculated and summed. Using the same percent estimates, a breakdown of the costs can be seen in Table 17.

Table 17. Double detention basin option. Present and Annual costs were calculated based on an inflation rate of 3% and a lifetime of 20 years.

Cost Item	Percent of Subtotal (%)	Cost
Base Cost		$53,070
Contractor OH & Profit	15	$7,961
Contractor MB&I	5	$2,654
Engineering	25	$13,268
Legal, Administrative	15	$7,961
Subtotal (constructed cost, incl. contractor, design, and legal/admin. Fees)		$84,913
Contingency	30	$25,474
Total Capital Cost		$110,387
Annual O&M		$5,519
Present Cost		$190,000
Annual Cost ($/year)		$13,000

4.4.3 Dredging and Culvert Maintenance Costs

It was determined that the base cost for sediment removal from Cashiers Creek will be about $12,300. This was calculated using an estimate of the volume of dredged material to be removed, an estimated cost per cubic yard of sediment, and an estimated mobilization and demobilization cost. The estimate is preliminary and does not include costs for permitting, and is based on a guide on environmental dredging from the EPA (USEPA 1994). Contractor, design, and legal/administrative fees were subsequently calculated from percent estimates of base costs. A 30% contingency was allocated for engineering detail and annual operational and maintenance (O&M). The annual cost was calculated using engineering economics formulas and breakdown of individual cost items for sediment removal can be seen in Table 18.

Table 18. Cost breakdown for sediment removal. Annual costs were calculated based on an inflation rate of 3% and a lifetime of 20 years.

Cost Item	Percent of Subtotal (%)	Cost
Subtotal (constructed cost)		$12,300
Contractor OH & Profit	15	$1,845
Contractor MB&I	5	$615
Engineering	10	$1,230
Legal, Administrative	5	$615
Subtotal (constructed cost, incl. contractor, design, and legal/admin. Fees)		$16,605
Contingency	30	$4,982
Total Capital Cost (Present Cost)		$22,000
Annual Cost ($/year)		$1,500

The cost for dredging the wetland was estimated at $28,600 using the same EPA guidelines on cost per cubic yard and mobilization/demobilization (USEPA 1994). The area of dredging was estimated using Google Earth: more survey is needed to determine the project area to a greater degree of accuracy. The estimate does not include surveying or permitting costs. Using the same percent estimates as the sediment removal for contractor, design, and legal/administrative fees, contingency, and annual O&M costs, the annual cost of wetland dredging was calculated. A breakdown of the costs can be seen in Table 19.

Table 19. Cost breakdown for wetland dredging. Annual costs were calculated based on an inflation rate of 3% and a lifetime of 20 years.

Cost Item	Percent of Subtotal (%)	Cost
Subtotal (constructed cost)		$28,600
Contractor OH & Profit	15	$4,290
Contractor MB&I	5	$1,430
Engineering	10	$2,860
Legal, Administrative	5	$1,430
Subtotal (constructed cost, incl. contractor, design, and legal/admin. Fees)		$38,610
Contingency	30	$11,583
Total Capital Cost (Present Cost)		$50,000
Annual Cost ($/year)		$3,400

Costs for cleaning the recommended culverts is estimated at $4,000 (Monroe County, Pennsylvania 2004). The North Carolina DOT is responsible for culvert maintenance, so this cost will likely not be the responsibility of Jackson County. All dredging and culvert maintenance activities should occur in the winter months (November to March) to avoid elevated water levels.

4.5.0 Maintenance Intervals for Stormwater Infrastructure

Regardless of how well components of a stormwater infrastructure are designed or constructed, they will not function as desired if maintenance is ignored. Stormwater BMP maintenance is an ongoing legal requirement for the duration of the BMP's lifetime, and must be conducted at the appropriate times by an appropriate professional and recorded for future reference.

BMP's are usually built, owned, and maintained by non-governmental entities. A signed and notarized Inspection and Maintenance Agreement (NCDENR 2007) must be included in the submission of any design plan of any BMP.

4.5.1 Basin Maintenance

Detention basins can vary significantly in terms of design, which can lead them to vary in inspection frequency and maintenance intervals. Table 20 shows what NCDENR BMP Manual recommends for inspection frequency according to Basin Type.

Table 20. Inspection Frequency for Detention Basins

Inspection Frequency	BMP
Monthly and within 24 hours after storms greater than 1 inch	Wet detention basin
Quarterly and within 24 hours after storms greater than 1 inch	Extended dry detention basin

Every Inspection and maintenance activity should be recorded and should include a minimum of date of inspection, condition of BMP, any maintenance work performed, and any issues to be fixed in the future. Other requirements may vary based on the BMP being maintained and inspected.

Maintenance tasks typically include removal of debris and trash, maintenance of mechanical components, or insect/pest control. Typically, the single most expensive cost of BMP maintenance, however is sediment removal. Dredging frequency is a function of the BMP site and design and could vary from every couple of years to 15-25 years. All factors being equal, wet detention basins are more difficult to maintain because wet sediment is harder and more expensive to remove than dry sediment.

Maintenance responsibility falls under the BMP owner: usually a private individual, corporation, or homeowner’s association.

Annual maintenance Costs can both be found in Section 5.4.1.

4.5.2 Culvert Maintenance

Culverts should be inspected regularly for blockage and damage. We recommend inspecting the culverts annually before the wet season. Annual inspections will allow for a logging of culvert conditions over time, and a schedule can be made for regular culvert cleaning based on how often they are blocked by sediment (CPRY Watershed Management Authority 2000). Regular inspections and maintenance for culverts are the responsibility of the North Carolina DOT.

4.6 Stormwater Engineering Evaluations

4.6.1 Detention Basin

Two options for detention basins were analyzed in Sections 4.3.1. Among these were the extended wet detention basin by itself and a combination of wet and dry detention basins at two different locations. A cost estimation on both were performed in Sections 4.4.1 and 4.4.2. Based on the cost and feasibility, single basin option can be selected, but for sediment removal efficiency and volume of detention, the double basin option offers more capacity for the system. Two detention basins add more resiliency to the system in case one is over capacity and fails to perform in extreme storm events.

4.6.2 Dredging

We recommend removing sediment from both Cashiers Creek and the wetland area north of Cashiers Lake. Dredging both areas will result in the best conditions for lowering the local water table and decreasing sedimentation in both areas. However, if the funds or means are not available (including permitting and resources for further surveying of the wetland area), we recommend first removing sediment from Cashiers Creek. If flooding subsides from dredging the creek alone, then dredging the wetland may not be necessary.

5. Roadway Engineering Design Approach & Recommendations

Flooding near Frank Allen Rd has the potential to cause damages to the road as well as prevent use of the road altogether. As a popular vacation destination and second home to many, Cashiers depends on its ability to maintain a relaxed and consistent transportation system in order to provide vacationers with a pleasant experience. However, flooding along Frank Allen Rd has the potential to disrupt the experience of Cashiers by restricting access to certain establishments.

5.1 Frank Allen Road Usage Characterization

Spanning from east to west of Cashiers, Frank Allen Rd provides the inhabitants of Cashiers access to many businesses and services including:

- Cashiers Post Office
- Albert Carlton Community Library
- Cashiers/Glenville Recreation Center
- Cashiers Aging Center: provides group meals, social activities, arts & crafts, and health promotion for seniors in the southern end of Jackson County
- Cashiers/Glenville Volunteer Fire Department: one of the four fire stations serving 135 square miles of Jackson County

The locations of these establishments are shown in Figure 36 and their usages are summarized in Table 21.

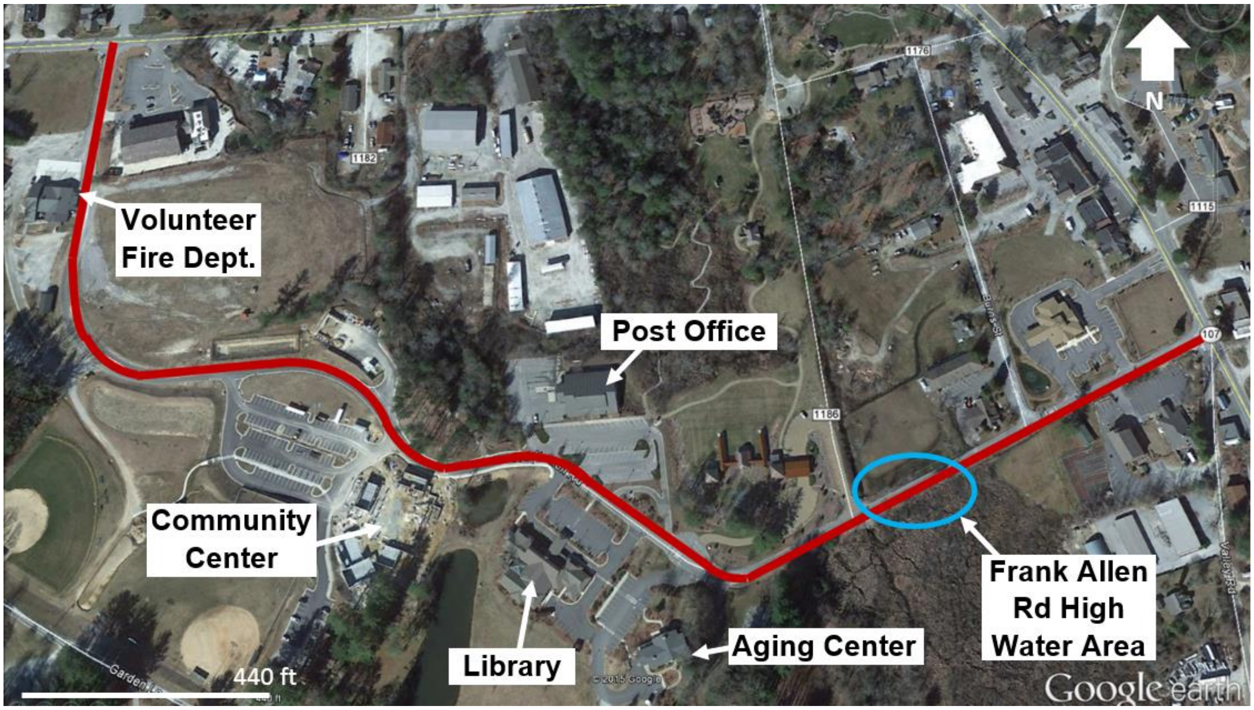

Figure 36. Frank Allen Rd establishment locations. Frank Allen Rd in red.

Table 21. Patrons served by Frank Allen Rd establishments.

Establishment	People Served
Cashiers Post Office	• 2,000 rented PO boxes • mail routes to 425 residents
Albert Carlton Community Library	• estimated 69,700 patrons for 2015
Cashiers Aging Center	• serves approximately 400 people
Cashiers/Glenville Recreation Center	• 1,030 members
Cashiers/Glenville Volunteer Fire Department	• serves up to approximately 4,000 people (non-peak vacation season residents) over an area of approximately 135 square miles

The peak visiting season for Cashiers brings a dramatic rise in the local population. From the end of May to the end of October, the population of Cashiers increases from approximately 2,000 to 10,000-15,000 people, approximately a 600% increase. Increased activity in the town establishments and traffic have been associated with this increase in population. While data is not available on the specific increase in trends of activity and traffic, eyewitnesses recount that traffic back ups occur more often during the peak season.

A one hour traffic survey on Frank Allen Rd during a weekday of April 2015 (non-peak season) counted 200 cars during 3:00 to 4:00 PM (including both east and west bound traffic). Assuming an increase in traffic flow rate equivalent to the increase in population from non-peak to peak season, a traffic flow rate of 1,200 cars/hour for 3:00 to 4:00 PM on a weekday during Cashiers peak season is estimated.

5.2 Road Usage Economic Value Analysis

The current stormwater system leaves the section of Frank Allen Rd over the double culverts vulnerable to both stormwater rushing over the road as well as standing water in the road and roadbed material (Figure 37). The loss of access through Frank Allen Rd due to flooding necessitates the use of a detour approximately one mile long. This detour uses NC Highway 107 for about 0.2 miles, US Route 64 for about 0.35 mi, and about 0.4 mi on the non-flooded portion of Frank Allen Rd. Additionally, this detour includes three sets of traffic lights, with all three being located at intersections with a highway (Figure 36).

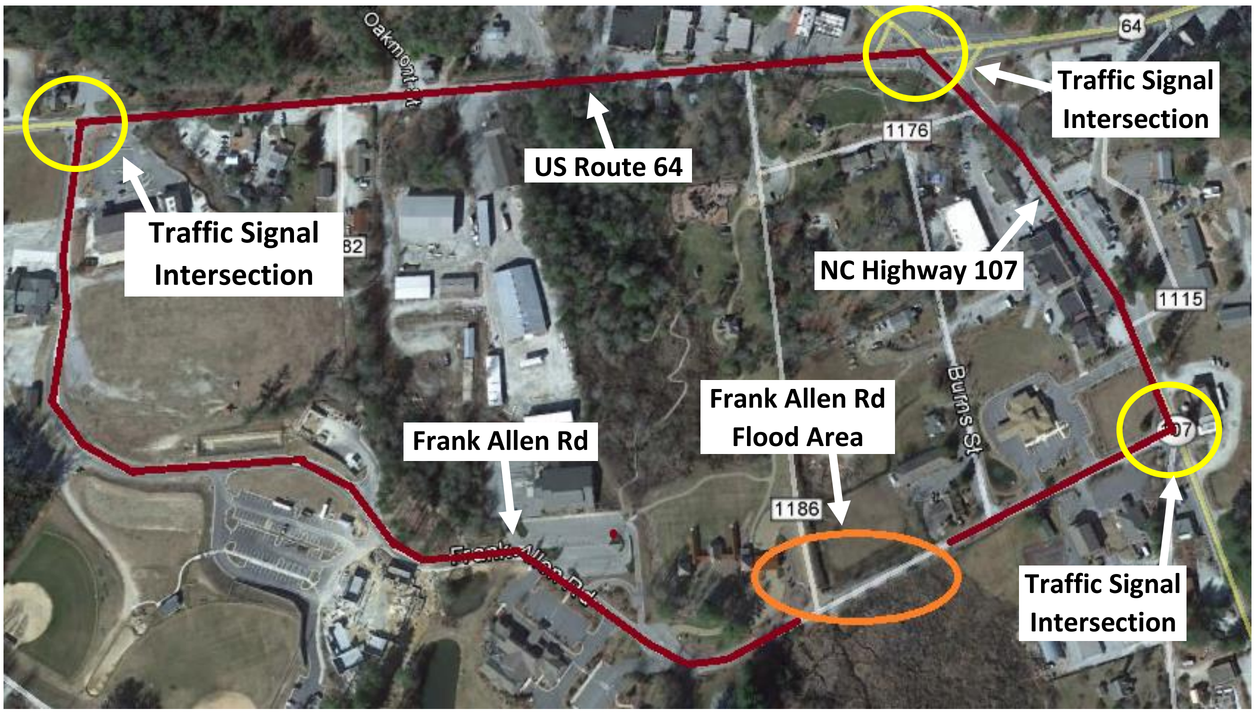

Figure 37. Frank Allen Rd Detour Route. The orange circle highlights the portion of Frank Allen Rd that would be affected by flooding and the high water table. The dark red line indicates the proposed detour that would allow access to both sides of Frank Allen Rd unaffected by flooding. The yellow circles outline traffic signal intersections.

5.2.1 Time Value Estimation

Assuming a traveler maintains an average speed of 35 mph during the mile-long detour and encounters a one minute wait time at each traffic light, a total detour time of about five minutes is estimated. A monetary value of time lost due to this detour has been calculated using guidelines and procedures established by the American Association of State Highway and Transportation Officials' User and Non-User Benefit Analysis for Highways, data from the 2009 National Household Transportation Survey (NHTS) Databook, and data on median income and wages (see Appendix I for more details). This monetary value is calculated by evaluating how the extra time spent taking the detour would have been spent in a productive manner and is intended to represent an opportunity cost associated with that lost time. These costs are listed in Table 22 below.

Table 22. Value of time estimates using AASHTO User and Non-User Benefit Analysis for Highways.

3:00-4:00 PM Traffic Along Frank Allen Rd	Traffic Flow Rate (cars/hr)	Total Delay Costs for Passenger Cars on Personal Travel ($)	Total Delay Costs for Passenger Cars on Business Travel ($)	Total Delay Costs for Passenger Cars ($)
Non-peak Traffic	200	430	43	473
Peak Traffic	1,200	2,600	260	2860

The value of time estimates listed above are then repeated for passenger cars on business travel. For business travel, the median income used in the previous Step 3 is replaced with the hourly employment cost (wages plus benefits). The total delay costs for personal and business travel and then summed to produce a cost in dollars per day of loss due to a detour. Using a Cashiers median household income of $42,700 ("Cashiers, NC Data & Demographics" 2014) an hourly employment cost of $33.33 (Bureau of Labor Statistics 2015), and the 200 cars/hour traffic flow rate found during a site visit, it was calculated that Cashiers residents traveling along Frank Allen Rd from 3:00 to 4:00 PM would lose $473 if flooding necessitated the use of the detour. Using 600% increase in traffic flow rate assumed earlier, monetary value of lost travel time during Cashiers' peak season was estimated to be approximately $2,860 for the 3:00 to 4:00 PM timeframe.

Afternoon storms during the summer have been recognized to cause the worst flooding. Assuming an increase in the afternoon traffic flow rate of 208 cars/hour (previously mentioned in section 5.1) that is proportional to the increase of population during the peak season would estimate approximately 1,300 cars/hour during the time when flooding is most likely to occur—summer afternoon storm events. This detour would then cause those 1,300 cars passing through Frank Allen Rd approximately 108 hours of lost time.

In addition to the lost time, the possibility of flooding closing down Frank Allen Rd near the culverts while a traffic accident or obstruction has occurred near the intersection of Frank Allen Rd and US Route 64 would prevent any access to the establishments listed in Table 21. This event could trap the Frank Allen Rd station of the Cashiers/Glenville Volunteer Fire Department. The Cashiers/Glenville Volunteer Fire Department serves an area of approximately 135 square miles with four stations. The temporary loss of the Frank Allen Rd station could strain the resources of the other stations by requiring them to serve a larger area. Additionally, a larger service area could cause longer response times, increasing public risk.

5.3 Roadway Repairs

Visible damages to Frank Allen Rd include longitudinal cracks (Figure 38) and scouring of the road shoulder (Figure 40). Amount of damages are summarized in Table 23.

Table 23. Summary of damages to Frank Allen Rd.

Type of Roadway Damage	Unit of Measurement	Amount of Damage	Notes
Longitudinal Cracks	linear foot of cracking	~100 ft	cracks are less than 0.5” in width
Road Shoulder Scour	linear foot of scouring	~20 ft	scour is about 2-3” deep and exposes the side of the road surface

5.3.1 Longitudinal Cracking

Longitudinal cracks appear parallel to the road centerline and can be due to various reasons—poor construction, extension of cracks in the underlying material, fatigue loading, etc. Because the majority of the cracks are not within the wheel path of the road, the longitudinal cracks at Frank Allen Rd are not likely to be load induced, and thus, poor construction is a likely cause of the formation of the crack (Lavin 2003).

Figure 38. Frank Allen Rd Longitudinal Cracks. Left photo shows a crack in the center of the lane, outside of the wheel-path. Right photo shows a crack along the lane striping, outside of the wheel-path. Average crack width < 0.5 inches.

The classification of the severity of the crack, and consequently the recommended treatment, depends on the mean width of the crack. As the longitudinal cracks along Frank Allen Rd are less than 0.5”, they can be classified as moderately severe cracks. Moderately severe longitudinal cracks have two available treatment options: clean and seal, rout and seal (Johnson 2000).

Clean and seal is the cheaper of the two options (see Table 24 for unit price comparison). The crack is cleaned before the placement of the sealant to ensure the best bond between the pavement and the sealant: cleaning is done using a high-pressure jet of air. Drying must also be done by using a hot air lance to reduce the amount of moisture in the crack, further ensuring the strongest possible bond. After the crack has been cleaned of debris and dried, asphalt sealant is poured into the crack to provide protection by reducing the entrance of water and incompressible material. Due to the potential for the pavement to experience temperature-related expansion effects, this method of repair is best conducted during spring and fall climates, when the air temperatures are moderately cool. Traffic must be rerouted to allow sealant material to cure; otherwise fine sand or toilet paper can be used to avoid sealant material from sticking to and being removed by passing tires. Asphalt sealant typically requires 24 hours to dry (Johnson 2000). See Figure 39 for clean and seal schematic.

Rout and seal takes the clean and seal procedure one step further with the addition of the creation of a reservoir for the sealant. The idea behind the reservoir is to improve the sealant’s adhesion to the pavement by cutting out more pavement beyond the crack to increase the bonding surface area as well as smooth the bonding surface (Johnson 2000). Similar to clean and seal, this method offers the pavement protection against water and incompressible material, but rout and seal provides a potentially better bonding between pavement and sealant at an increased cost for excavating the reservoir. Infiltration of water into the pavement can cause further damages. Before the excavation of the reservoir, cleaning and drying are accomplished using a high-pressure jet of air and a hot air lance, respectively. Due to the potential for the pavement to experience temperature-related expansion effects, this method of repair is best conducted during spring and fall climates, when the air temperatures are moderately cool. Traffic must be rerouted to allow sealant material to cure; otherwise fine sand or toilet paper can be used to avoid sealant material from sticking to and being removed by passing tires. Asphalt sealant typically requires 24 hours to dry (Johnson 2000). See Figure 38 for rout and seal schematic.

Table 24. Frank Allen Rd repair options.

Repair Technique	Protection Against Water?	Reapplication Interval	Unit Price Per Linear Ft
Clean and Seal	• reduces or prevents water and incompressible material from entering the pavement	• annually up to three years	• $0.1 to $0.3
Rout and Seal	• reduces or prevents water and incompressible material from entering the pavement and provides some protection against infiltration	• four years	• $0.5 to $0.85

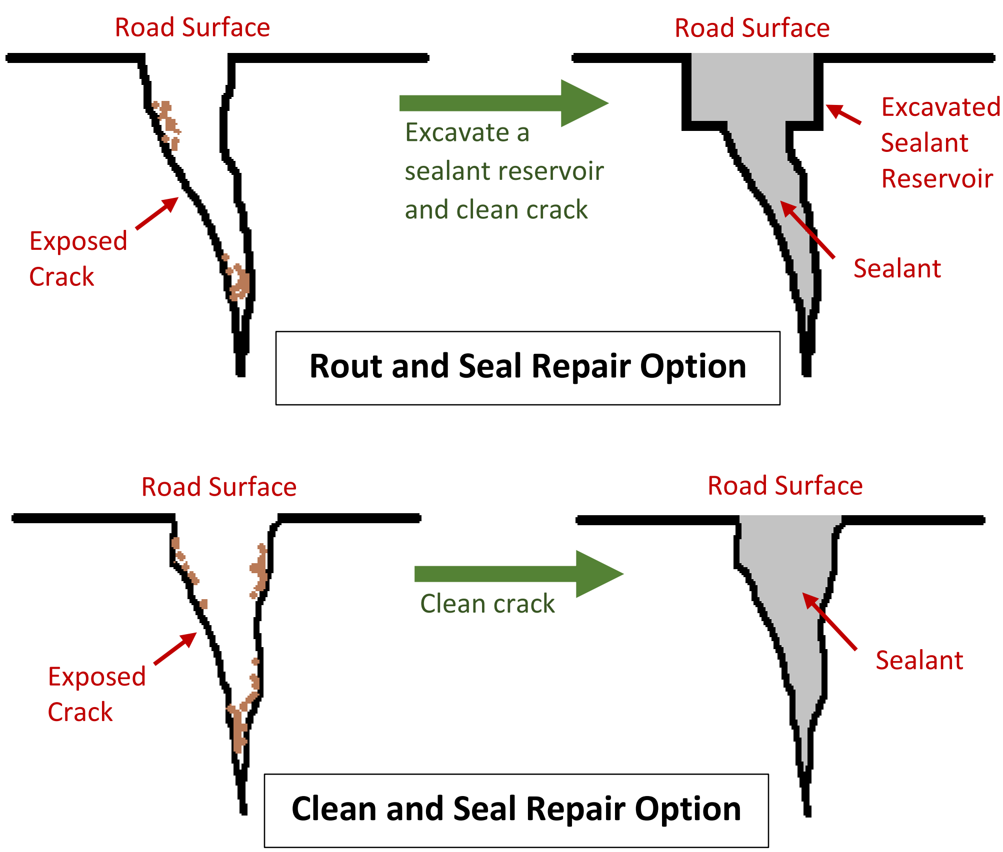

Figure 39. Frank Allen Rd repair method schematic.

5.3.2 Road Shoulder Scour

Scour of the road shoulder has taken place where Frank Allen Rd has been repeatedly overtopped by flows with velocities fast enough to erode the shoulder material (Figure 39).

Figure 40. Frank Allen Rd Road Shoulder Material Scour highlighted by the red line, approximately 20 feet in length. Scoured area appears as shadow to the left of the red line.

Smaller sized riprap could be put into place to mitigate further scouring. The exposed soil would need to be covered with a geotextile layer to secure the sediments, then the riprap can be placed on top of the geotextile layer. This option has not been evaluated as a solution to the scour. Ideally, this scour should be protected against once the sedimentation basins are in place.

5.3.3 Repair Cost Estimations

In addition to the costs to repair the cracks listed in Table 23, several costs are associated with maintaining the road and traffic safety during and after the repairs. These additional costs include:

- Thermoplastic lane striping to replace lane striping affected by the cracks: $1.20/linear ft.
- Traffic education campaign to inform the community about road closures: varies.

If the option to halt traffic flow over the area being repaired is taken, detour signs and a public service announcement campaign would be needed. Additional costs specific towards this options are as follows:

- Detour signs to inform and direct drivers: $200/sign, it is estimated that approximately five signs will be needed to inform drivers of the detour
- Traffic education campaign to inform the community about road closure and detour (public service announcements, flyers, public hearings, etc.): varies.

If it is desired to keep Frank Allen Rd open during the repairs, the road speed should be lowered and police enforcement to ensure lower traffic speeds in order to protect the newly-placed sealants.

- Speed Limit Signing - $200/sign
- Police enforcement - $75/hour
- Traffic education campaign to inform the community about road closure and detour (public service announcements, flyers, public hearings, etc.): varies.

Tables 25 and 26 display the costs associated with each repair method and the option of closing or not closing Frank Allen Rd during repairs.

Table 25. Costs associated per reapplication interval with the option to close down Frank Allen Rd during road repairs.

Item	Unit Price	Quantity	Cost
Clean and Seal	$0.1-0.3/lin.ft.	100 ft	$20
		OR	
Rout and Seal	$0.5-0.85/lin.ft.	100 ft	$67.5
Thermoplastic Road Line Striping	$1.20/lin. ft.	40 ft.	$48
Detour Signs	$200/sign	5 signs	$1000
Traffic Education Campaign	Varies	Varies	Varies
		Subtotal w/ Clean & Seal	$1068 + traffic education campaign
		Subtotal w/ Rout & Seal	$1115.5 + traffic education campaign

Table 26. Costs associated per reapplication interval with the option to keep Frank Allen Rd open during road repairs.

Item	Unit Price	Quantity	Cost
Clean and Seal	$0.1-0.3/lin.ft.	100 ft	$20
OR			
Rout and Seal	$0.5-0.85/lin.ft.	100 ft	$67.5
Speed Limit Signing	$200/sign	2	$400
Police Enforcement	$75/hour	24 hours	$1800
Traffic Education Campaign	Varies	Varies	Varies
		Subtotal w/ Clean & Seal	$2220 + traffic education campaign
		Subtotal w/ Rout & Seal	$2267.5 + traffic education campaign

Assuming the cost of a typical traffic education campaign is $100, closing the road for repairs while using the rout and seal method is found to be the least expensive option. Table 27 summarizes the cost of each repair method under each repair scenario into an annual value and present value.

Table 27. Summary of road repair cost estimations for a time span of 20 years. Inflation rate is assumed to be 3%.

Time Period	close road for repairs		keep road open	
	clean and seal	rout and seal	clean and seal	rout and seal
Annual Value ($/yr)	$378	$291	$751	$566
Present Value ($)	$5,622	$4,322	$11,167	$8,419

5.4 Roadway Engineering Evaluations

Due to the potential of flooding, we recommend that rout and seal be used to repair the longitudinal cracks on Frank Allen Rd in order to provide the pavement with some protection against infiltration. Clean and seal offers inferior protection against infiltration. Moisture infiltrating the road cracks into the pavement can cause further damage to the pavement. We also recommend that the road be closed during the repair activities to ensure the best possible bond in order to take advantage of the longer reapplication interval of the rout and seal method. Closing the road while performing the rout and seal repair method is also the least expensive option.

6. Final Design and Recommendations

6.1 Final Cost Estimation

The present and annual cost of the two feasible options are shown below in Table 28 and Table 29. Complete cost estimations for the different components were calculated in sections 4.4 and 5.4.

Table 28. Total Present and Annual Cost of Single Basin Solution

Component	Present Cost	Annual Cost
Sediment Removal	$21,587	$1,451
Single Basin	$130,626	$8,780
Wetland Dredging	$50,193	$3,374
Road Repairs	$5,622	$378
Total	$208,000	$14,000

Table 29. Total Present and Annual Cost of Double Basin Solution

Component	Present Cost	Annual Cost
Sediment Removal	$21,587	$1,451
Double Basin	$192,500	$12,939
Wetland Dredging	$50,193	$3,374
Road Repairs	$5,622	$378
Total	$270,000	$18,000

6.2 Final Scheduling

Construction scheduling sets the construction timeline of the project components such that impact on the surrounding area is minimized and utility of the project components are maximized with respect to each other. Addressing issues downstream first facilitates the flow of the water upstream of the stormwater system, therefore it is recommended to begin constructing solutions that address issues farthest downstream and work in the upstream direction. Tables 30 and 31 illustrate the construction scheduling of the Single Basin Solution and the Double Basin Solution.

Table 30. Single Basin Solution Construction Scheduling

Task	Week Number							
	1	2	3	4	5	6	7	8
Wetland Dredging	■	■						
Stream Dredging		■						
Culvert Cleaning		■						
Wet Basin			■	■	■	■	■	■
Road Repairs								■

Table 31. Double Basin Solution Construction Scheduling

Task	Week Number							
	1	2	3	4	5	6	7	8
Wetland Dredging								
Stream Dredging								
Culvert Cleaning								
Wet Basin								
Dry Basin								
Road Repairs								

6.3 Final Evaluation

Our final evaluations components include stormwater detention basins, sediment removal, culvert maintenance, and roadway repairs. We recommend implementing all of these components to achieve the best results of flooding mitigation. Leaving the flooding issues in Cashiers unaddressed is unadvisable as this poses unnecessary risk to the community.

Dredging of the wetland and of Cashiers Creek to remove excess sediment should occur first. We recommend conducting dredging over both areas, though dredging Cashiers Creek alone may be sufficient to improve flow and lower the water table. Sediment removal should occur every four years. This time period may be adjusted based on need. Culvert maintenance can occur in tandem with sediment removal to minimize disposal costs.

Construction of the detention basin(s) will follow dredging operations. Two options are available for the implementation of this feature: a single wet detention basin, or a combination of one wet and one dry detention basin. The single basin solution is more feasible and cost effective. Alternatively, the two basin solution requires less time, is more efficient in sediment removal, and offers greater capacity for extreme storm events.

Repairs of Frank Allen Road will be the last component to be executed. To provide protection from further damage to the road, we recommend repairing the longitudinal cracks with the rout and seal method. The clean and seal method will not provide adequate protection against infiltration. The road should be closed during repair activities to ensure the best repair possible with the rout and seal method.

Appendix A

Topography Survey

Disclaimer:

Larger paper was not available for the printed copies of this report. Topographic survey is better viewed as a digital copy.

NOTES:

1. ELEVATIONS ARE BASED ON GPS OBSERVATIONS-NAVD88 BY ALLIANCE SURVEYING CO. 3-12-2015.
2. UNDERGROUND UTILITIES AND IMPROVEMENTS NOT READILY VISIBLE WERE NOT LOCATED.
3. BEARING ORIENTATION IS BASED ON NAD83(2011).
4. THIS MAP WAS PREPARED SOLELY FOR THE PURPOSE OF SHOWING TOPOGRAPHY AND EXISTING SITE CONDITIONS. IT IS NOT INTENDED TO BE USED FOR DEED PREPARATIONS, REAL ESTATE CONVEYANCE OR RECORDING.
5. THIS MAP DOES NOT REPRESENT A BOUNDARY SURVEY!
6. BOUNDARY INFORMATION HAS NOT BEEN CHECKED OR VERIFIED.
7. DEED INFORMATION HAS NOT BEEN CHECKED OR VERIFIED.
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13. PROPERTY HAS NOT BEEN INSPECTED FOR WETLANDS OR FLOOD HAZARDS.

N

GRID North
NAD 83(2011)
NAVD 88 (Geoid 12A)

Culvert Table

Culvert ID	Size/Material	Invert In	Invert Out	Remarks
C-A	60" CMP	3471.95'	3472.11'	Outlet is partially obstructed by silt.
C-B	60" CMP	3471.76'	3472.19'	Outlet is partially obstructed by silt.
C-C	84" x 56" CMP	3472.68'	3472.46'	The culvert is mostly submerged. Possibly oval or "D" shaped but unable to determine exactly.
C-D	84" x 56" CMP	3472.58'	3472.62'	The culvert is mostly submerged. Possibly oval or "D" shaped but unable to determine exactly.
C-E	24" CMP	3476.77'	3476.09'	Remarks
C-F	24" CMP	3479.52'	3479.47'	Remarks
C-G	84" x 56" CMP	3478.57'	3478.36'	The culvert is mostly submerged. Possibly oval or "D" shaped but unable to determine exactly.
C-H	72" CMP	3480.61'	3481.25'	Outlet is partially silted. Elevation may not represent the true invert of the pipe.
C-I	72" CMP	3480.45'	3480.46'	Outlet end of culvert is heavily silted.

Cashiers Lake elevations at the dam.

Point	Northing	Easting	Elev.	Description
100	518497.54	773982.53	3471.32	WATER LEVEL
101	518490.83	774033.51	3470.24	CHANNEL BOTTOM
103	518466.23	773671.82	3471.26	RISER TOP

LEGEND :

- 5 FOOT CONTOUR
- 1 FOOT CONTOUR
- Asphalt Road
- Gravel Road
- Existing Creek
- Tree Line
- Drainage Structure
- Existing Fence Line
- Shrub / Tree
- Light Pole
- Utility Pole
- Sign
- Spot ELevation
- Ground Aerial Target

Cashiers Creek Site

INDEPENDENT MAPPING COMPANY Job No. 2015–021

This map was produced by photogrammetric methods from aerial photography in accordance with National Map Accuracy Standards for 1"=40' scale mapping with 1' contours as published by the USGS. Planimetric and topographic features that lie in areas delineated as "OBSCURE" may not adhere to National Map Accuracy Standards.

Ninety–five (95) percent of all contours on clear unobscured ground will be correct to within one–half (1/2) of the contour interval. The remaining five (5) percent will not exceed error of more than the contour interval.

Dashed–line contours shall have an accuracy with respect to true elevation of one contour interval or one–fourth (1/4) the average height of the ground cover, whichever is greater.

Ninety (90) percent of all spot elevations on clear unobscured ground will be correct to within twenty–five (25) percent of the specified contour interval, and none will be in error by more than fifty (50) percent of that contour interval.

Ninety–five (95) percent of all visible planimetric features shall be within one–fortieth (1/40) of an inch, at map scale, of their correct coordinate positions. No such well defined features shall be in error by more than one–twentieth (1/20) of an inch.

Contour Interval: 1 Foot
Date of Photography: March 7, 2015
Scale of Photography: 1"=330'

I, R. JOEL JOHNSON, CERTIFY THAT THIS PROJECT (DRAINAGE STRUCTURES, AERIAL TARGETS, SPOT ELEVATION ALONG FRANK ALLEN ROAD AND A PORTION OF CASHIERS CREEK ONLY)WAS COMPLETED UNDER MY DIRECT AND RESPONSIBLE CHARGE FROM AN ACTUAL GROUND SURVEY (CLASS A) MADE UNDER MY SUPERVISION; THAT THE SURVEY WAS COMPLETED ON MARCH 30, 2015; THAT THE CONTOURS SHOWN AS BROKEN LINES MAY NOT MEET THE STATED STANDARD; AND ALL COORDINATES ARE BASED ON NAD 83(2011) & NAVD 88. THIS IS NOT A CERTIFICATION OF BOUNDARIES, RIGHT-OF-WAYS, RESERVATIONS OR RESTRICTIONS OR LACK THEREOF. THIS TOPOGRAPHIC SURVEY SHALL NOT BE USED FOR RECORDING, CONVEYANCES OR REAL ESTATE SALES. IMC IS RESPONSIBLE FOR ALL CONTOURS AND DEPICTION OF ALL ABOVE GROUND STRUCTURES.

This document originally issued and sealed by
R. Joel Johnson, PLS L–3882, 03–30–2015
This media shall not be considered a certified document.

R. JOEL JOHNSON, P.L.S. DATED 03-30-2015
REGISTRATION NUMBER L-3882

100' 50' 0 100' 200'
Scale In Feet
Graphic Scale 1" = 100'

Topographic Survey for :
Jackson County Planning

Showing :
A 66 Acre aerial topographic survey of a portion of Cashiers N.C.

Location :
Frank Allen Road
Cashiers, NC
Cashiers Township
Jackson County, North Carolina

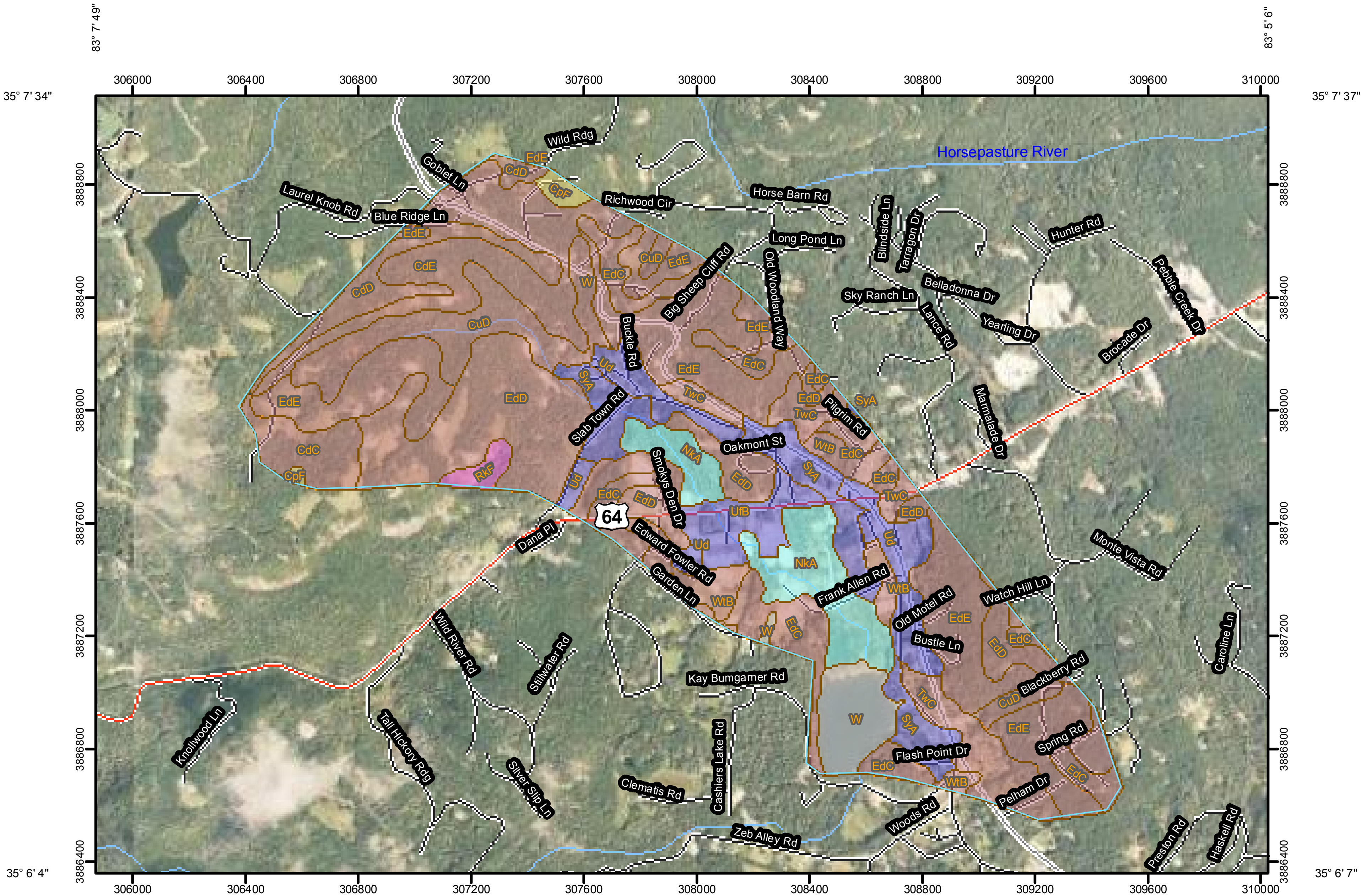
Prepared By:
Joel Johnson Land Surveying, Inc.
Corporate License C - 2436

701 Old Settlement Road
Sylva, North Carolina 28779
Tel. 828-586-6488
E-mail rjjohnson31@hotmail.com
joel@joeljohnsonlandsurveying.com

Field Crew	Johnson-Robison-IMC	Drawing Scale	1" = 100'
Field Book	15-01	PIN	---- ---- ----
Data File	15-017A	Title Source	---- ----
Dwg. File	15-017T	Survey Date	3-2015
Project Number	15-017	Plot Date	3-30-2015

Appendix B

Soil Survey



Map Scale: 1:19,700 if printed on A size (8.5" x 11") sheet.

Meters
0 250 500 1,000 1,500

Feet
0 1,000 2,000 4,000 6,000

N

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Map Units

Soil Ratings

A

A/D

B

B/D

C

C/D

D

Not rated or not available

Political Features

Cities

Water Features

Oceans

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

MAP INFORMATION

Map Scale: 1:19,700 if printed on A size (8.5" × 11") sheet.

The soil surveys that comprise your AOI were mapped at 1:12,000.

Please rely on the bar scale on each map sheet for accurate map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: UTM Zone 17N NAD83

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Jackson County, North Carolina
Survey Area Data: Version 7, Apr 17, 2009

Date(s) aerial images were photographed: 9/21/2006

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Hydrologic Soil Group— Summary by Map Unit — Jackson County, North Carolina				
Map unit symbol	**Map unit name**	**Rating**	**Acres in AOI**	**Percent of AOI**
CdC	Chandler gravelly fine sandy loam, 8 to 15 percent slopes	A	19.1	2.6%
CdD	Chandler gravelly fine sandy loam, 15 to 30 percent slopes	A	25.0	3.4%
CdE	Chandler gravelly fine sandy loam, 30 to 50 percent slopes	A	64.2	8.8%
CpF	Cleveland-Chestnut-Rock outcrop complex, windswept, 50 to 95 percent slopes	C	4.7	0.6%
CuD	Cullasaja-Tuckasegee complex, 15 to 30 percent slopes, stony	A	50.0	6.8%
EdC	Edneyville-Chestnut complex, 8 to 15 percent slopes, stony	A	63.9	8.7%
EdD	Edneyville-Chestnut complex, 15 to 30 percent slopes, stony	A	122.9	16.7%
EdE	Edneyville-Chestnut complex, 30 to 50 percent slopes, stony	A	138.1	18.8%
NkA	Nikwasi fine sandy loam, 0 to 2 percent slopes, frequently flooded	B/D	46.4	6.3%
RkF	Rock outcrop-Cleveland complex, windswept, 30 to 95 percent slopes	D	5.3	0.7%
SyA	Sylva-Whiteside complex, 0 to 2 percent slopes	B	45.0	6.1%
TwC	Tuckasegee-Whiteside complex, 8 to 15 percent slopes	A	40.3	5.5%
Ud	Udorthents, loamy	B	39.3	5.3%
UfB	Udorthents-Urban land complex, 0 to 5 percent slopes, rarely flooded	B	20.9	2.9%
W	Water		23.0	3.1%
WtB	Whiteside-Tuckasegee complex, 2 to 8 percent slopes	A	26.0	3.5%
Totals for Area of Interest			**734.0**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: *None Specified*

Tie-break Rule: Lower

Appendix C

Dam Report Calculations

RUNOFF CURVE NUMBER DATA

Soil/Surface Description	CN	Area acres	Impervious Adjustment %C	Impervious Adjustment %UC	Adjusted CN
Water	98	24.530			98.00
Residential 2 acre	65	42.450			65.00
Residential 1 acre	68	137.590			68.00
Commercial and Business	92	78.640			92.00
Open Space Good	61	36.780			61.00
Woods Good	55	351.817			55.00
COMPOSITE AREA & WEIGHTED CN --->		671.807			64.52 (65)

```
------------------------------------------------------------------------
Tc Equations used...
------------------------------------------------------------------------


==== SCS TR-55 Sheet Flow ==============================================

     Tc = (.007 * ((n * Lf)**0.8)) / ((P**.5) * (Sf**.4))

     Where:  Tc = Time of concentration, hrs
             n  = Mannings n
             Lf = Flow length, ft
             P  = 2yr, 24hr Rain depth, inches
             Sf = Slope, %


==== SCS TR-55 Shallow Concentrated Flow ===============================

     Unpaved surface:
     V  = 16.1345 * (Sf**0.5)

     Paved surface:
     V  = 20.3282 * (Sf**0.5)

     Tc = (Lf / V) / (3600sec/hr)

     Where:  V  = Velocity, ft/sec
             Sf = Slope, ft/ft
             Tc = Time of concentration, hrs
             Lf = Flow length, ft
```

==== SCS Channel Flow ==

```
R  = Aq / Wp
V  = (1.49 * (R**(2/3)) * (Sf**-0.5)) / n

Tc = (Lf / V) / (3600sec/hr)

Where:  R  = Hydraulic radius
        Aq = Flow area, sq.ft.
        Wp = Wetted perimeter, ft
        V  = Velocity, ft/sec
        Sf = Slope, ft/ft
        n  = Mannings n
        Tc = Time of concentration, hrs
        Lf = Flow length, ft
```

Appendix D

Sieve Analysis

	Sample	Tray Number	Tray Weight (g)	Wet Sample Weight (g)	Dry Sample Weight (g)	Final Dry Weigth to be Sieved
Day 1	Carolina Sm	15	347.83	922.4	891.88	891.90
	W1 (downs	III	348.05	716.38	538.32	538.32
	W8 (downs	6	346.76	988.84	752.96	753.26
	W2 (upstre	12	346.67	851.47	656.19	656.42
	W9 (upstre	1	366.80	943.59	372.80	372.44
	Village Gre	5	356.87	634.14	633.07	633.30
Day 2	W4 (stream	15	347.83	1237.97	1132.89	1132.75
	W6 (downs	III	348.05	1428.70	1107.67	1107.56
	D1 (side of	6	346.76	1317.08	1077.82	664.50
	D2 (side of	12	346.67	1034.56	1031.13	1031.04
	W7 (culvert	1	366.80	1143.11	972.42	972.42
	W3 (stream	5	356.87	1453.07	1165.25	1165.25
	W5 (Stream	7	355.70	2142.85	1473.89	1472.67
	W9 (organi	TPL3	346.97	746.93	311.72	311.72
	evil sample combined weights					677.74

set 1

sieve	weight (g)	
pan	348.7	
1/4	588.3	593
6	504.7	504.4
10	470.9	
12	423.2	
16	415.6	
30	405.2	405.2
40	499.3	
60	341.6	
70	364	
100	364.9	
140	351.4	
200	506.8	

Sieve Analysis

Sample nur W3
Sample We 506.60 g

Sieve Numl	Sieve Size (	Sieve Weig	Weight of S	Weight of S	Cumulative	Weight Pas	% Passing
10	2	470.9	563.8	92.9	92.9	413.70	81.66%
16	1.19	415.6	575.4	159.8	252.7	253.90	50.12%
40	0.42	499.3	739.3	240	492.7	13.90	2.74%
70	0.21	364	374.8	10.8	503.5	3.10	0.61%
200	0.074	506.8	508.1	1.3	504.8	1.80	0.36%
pan		348.7	349	0.3	505.1	1.50	0.30%

Sample nur W7
Sample We 518.70 g

Sieve Numl	Sieve Size	Sieve Weig	Weight of S	Weight of S	Cumulative	Weight Pas	% Passing
1/4	6.35	593	612.5	19.5	19.5	499.20	96.24%
6	3.36	504.7	556.6	51.9	71.4	447.30	86.23%
10	2	470.7	557.1	86.4	157.8	360.90	69.58%
16	1.19	415.6	519.1	103.5	261.3	257.40	49.62%
40	0.42	499.4	694.6	195.2	456.5	62.20	11.99%
70	0.21	364	416.3	52.3	508.8	9.90	1.91%
pan		345.8	355	9.2	518	0.70	0.13%

Sample nur W5
Sample We 506.40 g

Notes
leaves
leaves

Sieve Numl	Sieve Size	Sieve Weig	Weight of S	Weight of S	Cumulative	Weight Pas	% Passing
10	2	471	547.8	76.8	76.8	429.60	84.83%
30	0.595	405.3	559.5	154.2	231	275.40	54.38%
50	0.297	455.8	621.1	165.3	396.3	110.10	21.74%
80	0.177	430.7	499.5	68.8	465.1	41.30	8.16%
140	0.105	351.5	380.3	28.8	493.9	12.50	2.47%
200	0.074	506.8	513.3	6.5	500.4	6.00	1.18%
pan		348.1	354.4	6.3	506.7	-0.30	-0.06%

misslabled

Sample nur W4
Sample We 1132.80 g

Notes
leaves
leaves
leave bits

Sieve Numl	Sieve Size (	Sieve Weig	Weight of S	Weight of S	Cumulative	Weight Pas	% Passing
1/4	6.35	592.9	896.2	303.3	303.3	829.50	73.23%
6	3.36	504.6	733.6	229	532.3	600.50	53.01%
10	2	470.7	648.3	177.6	709.9	422.90	37.33%
16	1.19	415.5	567.9	152.4	862.3	270.50	23.88%
40	0.42	499.2	687.5	188.3	1050.6	82.20	7.26%
70	0.21	363.8	410.2	46.4	1097	35.80	3.16%
pan		345.7	381.7	36	1133	-0.20	-0.02%

Sample nur W6
Sample We 1101.80 g

Notes
some leave

Sieve Numl	Sieve Size	Sieve Weig	Weight of S	Weight of S	Cumulative	Weight Pas	% Passing
20	0.841	495.2	549.5	54.3	54.3	1047.50	95.07%
30	0.595	405.2	486.7	81.5	135.8	966.00	87.67%
50	0.297	456	1030.3	574.3	710.1	391.70	35.55%
80	0.177	430.7	756.5	325.8	1035.9	65.90	5.98%
140	0.105	351.5	409.2	57.7	1093.6	8.20	0.74%
200	0.074	506.9	511.5	4.6	1098.2	3.60	0.33%
pan		347.8	350.6	2.8	1101	0.80	0.07%

Sample nur W9 (organic)
Sample We 676.70 g

Notes
almost all l
leaves

Sieve Numl	Sieve Size	Sieve Weig	Weight of S	Weight of S	Cumulative	Weight Pas	% Passing
20	0.841	495	551.6	56.6	56.6	620.10	91.64%
30	0.595	405.3	439.5	34.2	90.8	585.90	86.58%
50	0.297	456.1	571.4	115.3	206.1	470.60	69.54%
80	0.177	431	545.1	114.1	320.2	356.50	52.68%
140	0.105	351.4	475.6	124.2	444.4	232.30	34.33%
200	0.074	506.9	576.3	69.4	513.8	162.90	24.07%
pan		347.7	510.4	162.7	676.5	0.20	0.03%

Sample nur Vill. Green
Sample We 633.30 g

Note
Sieves busted apart :(

Sieve Numl	Sieve Size	Sieve Weig	Weight of S	Weight of S	Cumulative	Weight Pas	% Passing
3/8	9.51	852.1					
1/4	6.35	593					
6	3.36	504.7	601.6	130	130		
10	2	471.6					
16	1.19	415.7					
pan		366.9					

Sample nur smokehouse parking
Sample We 891.80 g

Sieve Numl	Sieve Size	Sieve Weig	Weight of $	Weight of $	Cumulative	Weight Pas	% Passing
3/8	9.51	851.7	1295.8	444.1	444.1	447.70	50.20%
10	2	471.1	645.7	174.6	618.7	273.10	30.62%
30	0.595	405.1	453.9	48.8	667.5	224.30	25.15%
50	0.297	455.8	501	45.2	712.7	179.10	20.08%
80	0.177	430.6	472.9	42.3	755	136.80	15.34%
140	0.105	351.3	400.5	49.2	804.2	87.60	9.82%
pan		347.7	435.8	88.1	892.3	-0.50	-0.06%

Sample nur W2
Sample We 656.20 g

Sieve Numl	Sieve Size	Sieve Weig	Weight of $	Weight of $	Cumulative	Weight Pas	% Passing
10	2	471	481.5	10.5	10.5	645.70	98.40%
20	0.841	495	671.4	176.4	186.9	469.30	71.52%
30	0.595	405.4	595	189.6	376.5	279.70	42.62%
50	0.297	455.7	718.7	263	639.5	16.70	2.54%
80	0.177	430.7	443.8	13.1	652.6	3.60	0.55%
140	0.105	351.4	353.7	2.3	654.9	1.30	0.20%
pan		347.7	349.1	1.4	656.3	-0.10	-0.02%

Sample nur W8
Sample We 752.90 g

Notes	Sieve Numl	Sieve Size	Sieve Weig	Weight of $	Weight of $	Cumulative	Weight Pas	% Passing
mostly leav	10	2	471	496.4	25.4	25.4	727.50	96.63%
some leave	20	0.841	458.3	666.4	208.1	233.5	519.40	68.99%
	30	0.595	412.6	602.4	189.8	423.3	329.60	43.78%
	40	0.42	499.5	658.7	159.2	582.5	170.40	22.63%
	50	0.297	455.7	549.6	93.9	676.4	76.50	10.16%
	100	0.149	350.3	411	60.7	737.1	15.80	2.10%
	pan		347.6	363.7	16.1	753.2	-0.30	-0.04%

Conversions

sieve	mm	in
pan		
1/4	6.35	0.25
6	3.36	0.132
10	2	0.0787
12	1.68	0.0661
16	1.19	0.0469
20	0.841	0.0331
30	0.595	0.0234
40	0.42	0.0165
50	0.297	0.0117
60	0.25	0.0098
70	0.21	0.0083
80	0.177	0.007
100	0.149	0.0059
140	0.105	0.0041
200	0.074	0.0029

Sample nur W1
Sample We 538.10 g

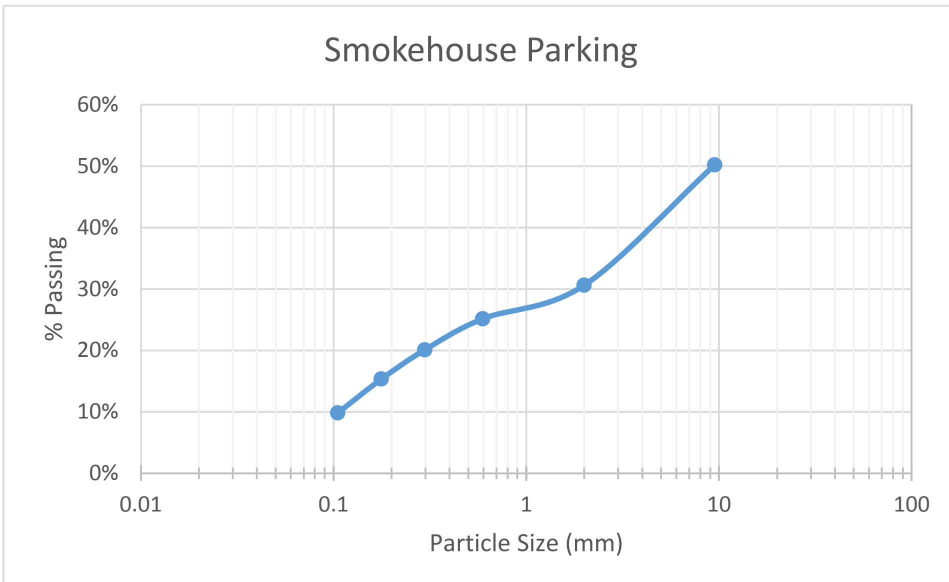
Notes	Sieve Numl	Sieve Size	Sieve Weig	Weight of $	Weight of $	Cumulative	Weight Pas	% Passing
leaves	10	2	470.8	517	46.2	46.2	491.90	91.41%
	20	0.841	495.2	641.3	146.1	192.3	345.80	64.26%
	30	0.595	405.2	513.9	108.7	301	237.10	44.06%
	40	0.42	499.6	604.7	105.1	406.1	132.00	24.53%
	50	0.297	455.7	529.8	74.1	480.2	57.90	10.76%
	100	0.149	367.4	414.2	46.8	527	11.10	2.06%
	pan		366.7	377.9	11.2	538.2	-0.10	-0.02%

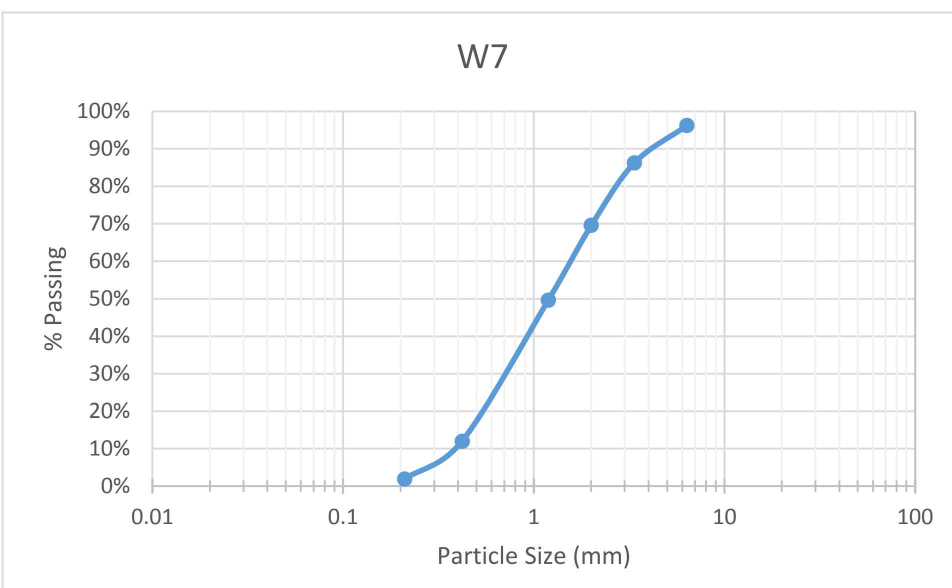
Sample nur D2
Sample We 1030.60 g

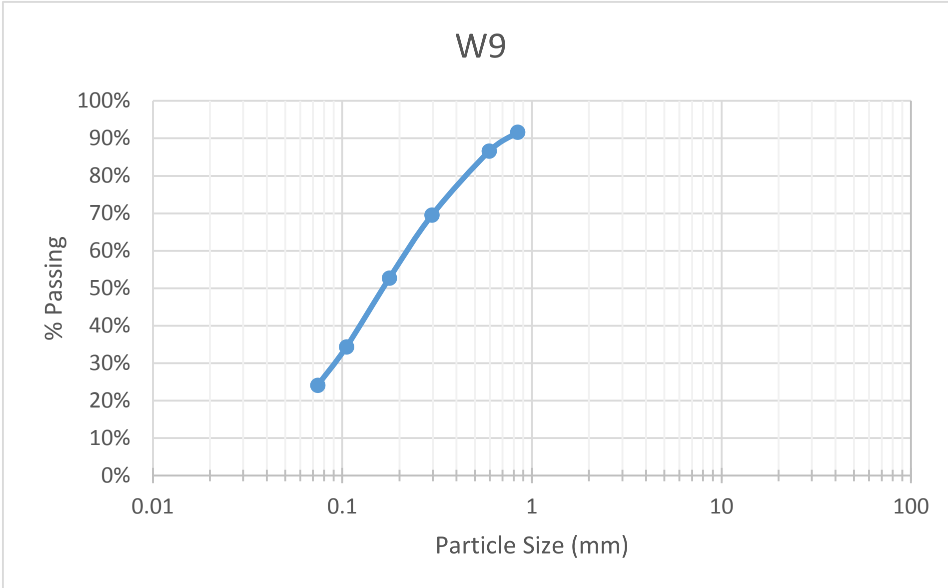
Sieve Numl	Sieve Size	Sieve Weig	Weight of $	Weight of $	Cumulative	Weight Pas	% Passing
10	2	470.8	700.9	230.1	230.1	800.50	77.67%
20	0.841	458.4	553.2	94.8	324.9	705.70	68.47%
30	0.595	412.6	471	58.4	383.3	647.30	62.81%
40	0.42	499.3	558.3	59	442.3	588.30	57.08%
50	0.297	455.6	533.9	78.3	520.6	510.00	49.49%
100	0.149	350.2	594.2	244	764.6	266.00	25.81%
pan		345.6	612.3	266.7	1031.3	-0.70	-0.07%

Sample nur D1
Sample We 664.20 g

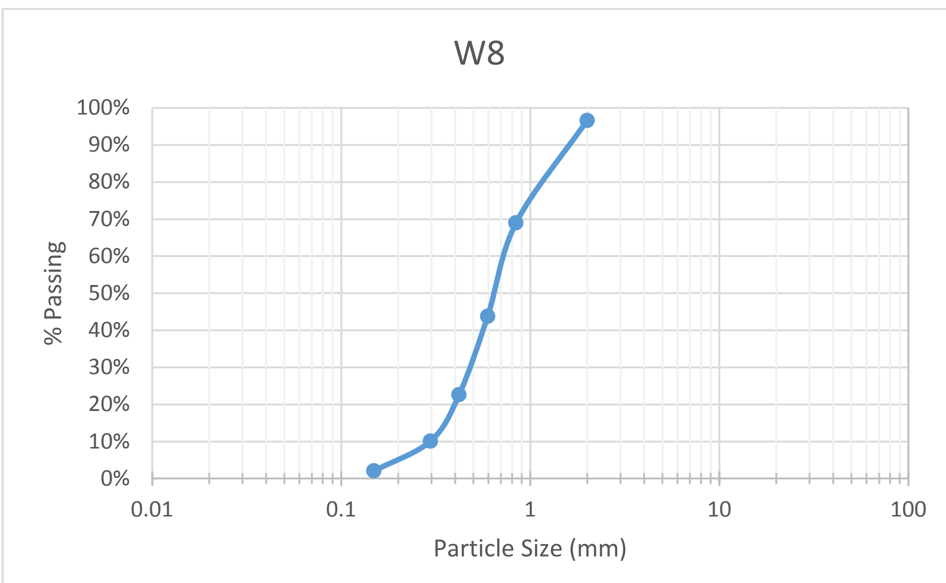
Sieve Numl	Sieve Size	Sieve Weig	Weight of $	Weight of $	Cumulative	Weight Pas	% Passing
10	2	470.7	513.6	42.9	42.9	621.30	93.54%
20	0.841	495.1	540.6	45.5	88.4	575.80	86.69%
30	0.595	405.2	460.1	54.9	143.3	520.90	78.43%
50	0.297	455.7	630.4	174.7	318	346.20	52.12%
80	0.177	430.6	553.5	122.9	440.9	223.30	33.62%
140	0.105	351.4	456.3	104.9	545.8	118.40	17.83%
pan		366.8	485.8	119	664.8	-0.60	-0.09%


Smokehouse Parking
60%
50%
40%
30%
20%
10%
0%
% Passing
0.01
0.1
1
10
100
Particle Size (mm)

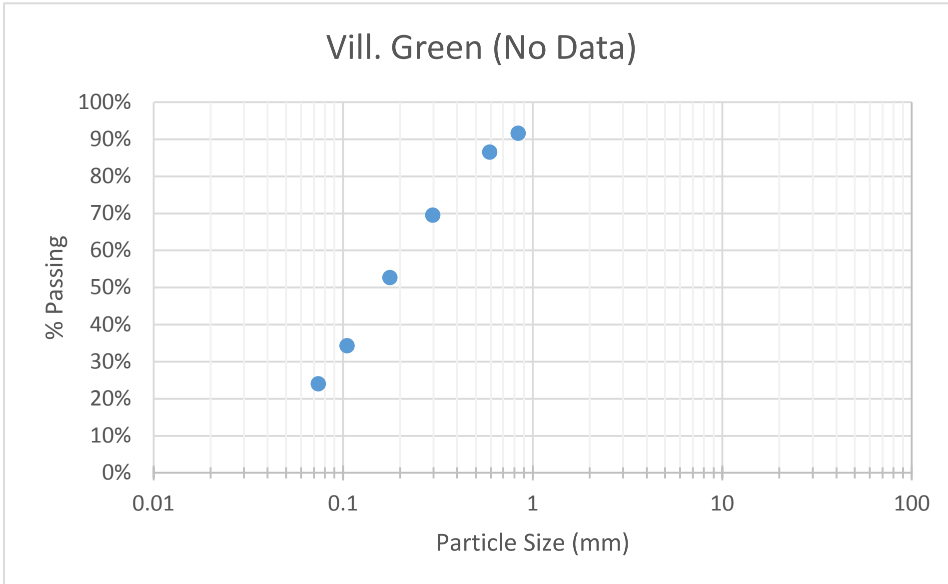

W7
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
% Passing
0.01
0.1
1
10
100
Particle Size (mm)



W9
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
% Passing
0.01
0.1
1
10
100
Particle Size (mm)



W8
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
% Passing
0.01
0.1
1
10
100
Particle Size (mm)


Vill. Green (No Data)
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20%
10%
0%
% Passing
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0.1
1
10
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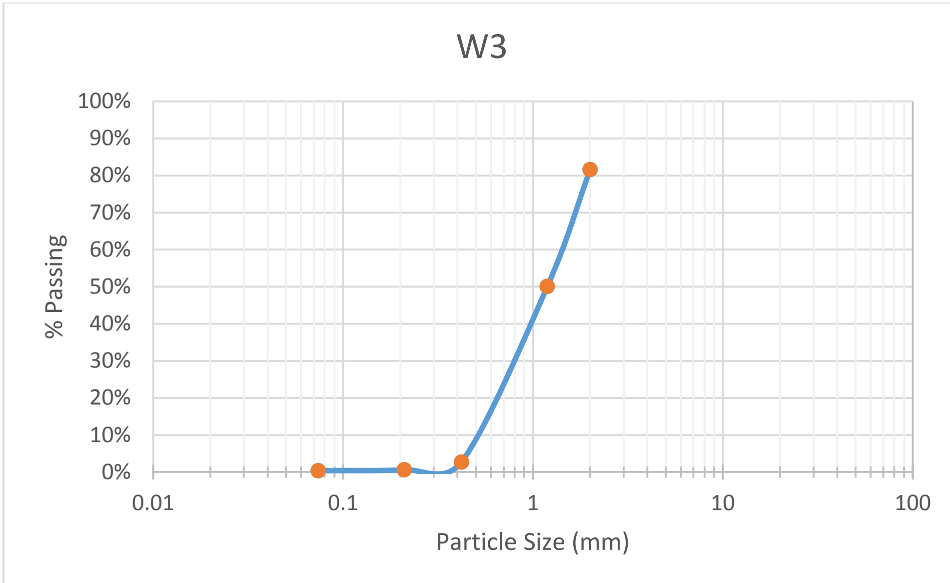
Particle Size (mm)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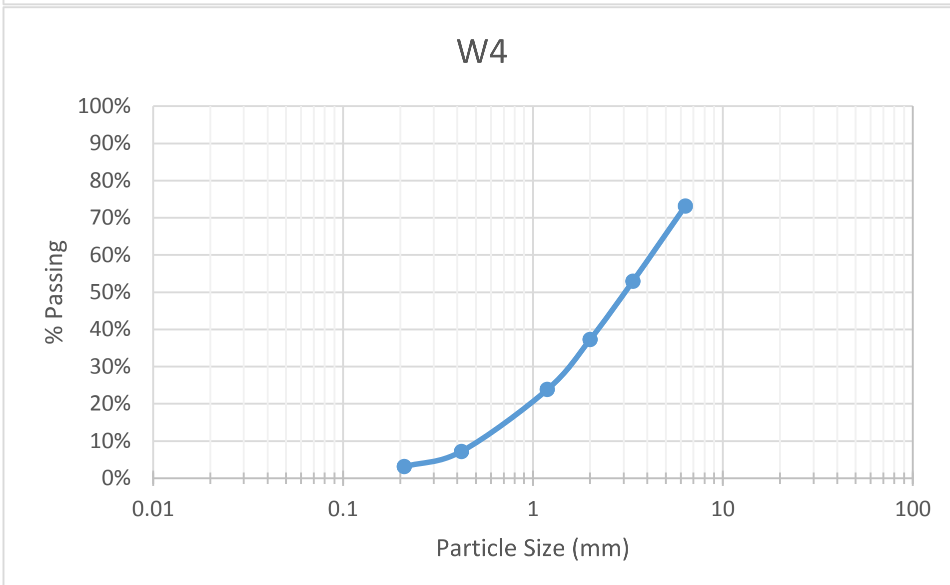
Particle Size (mm)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30%
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10%
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% Passing
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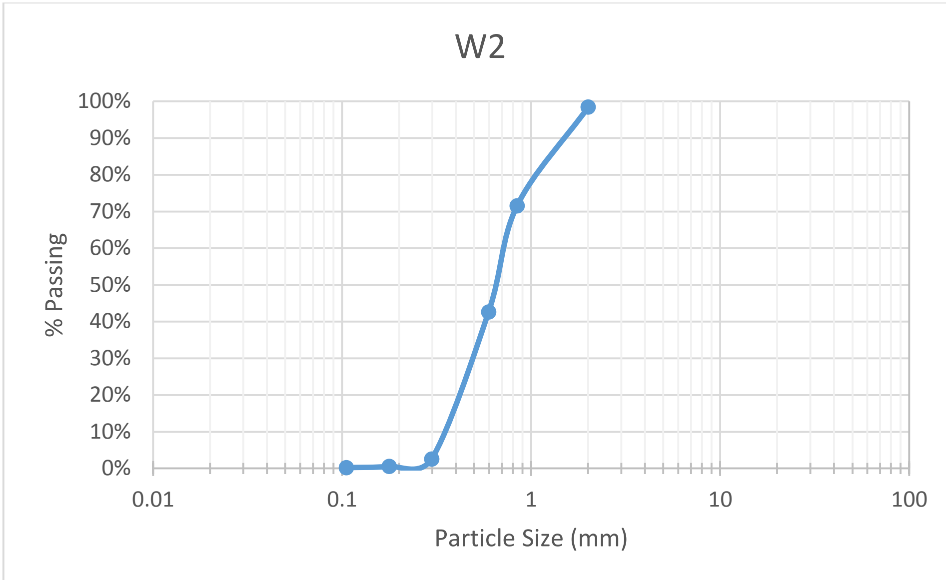
1
10
100
Particle Size (mm)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% Passing
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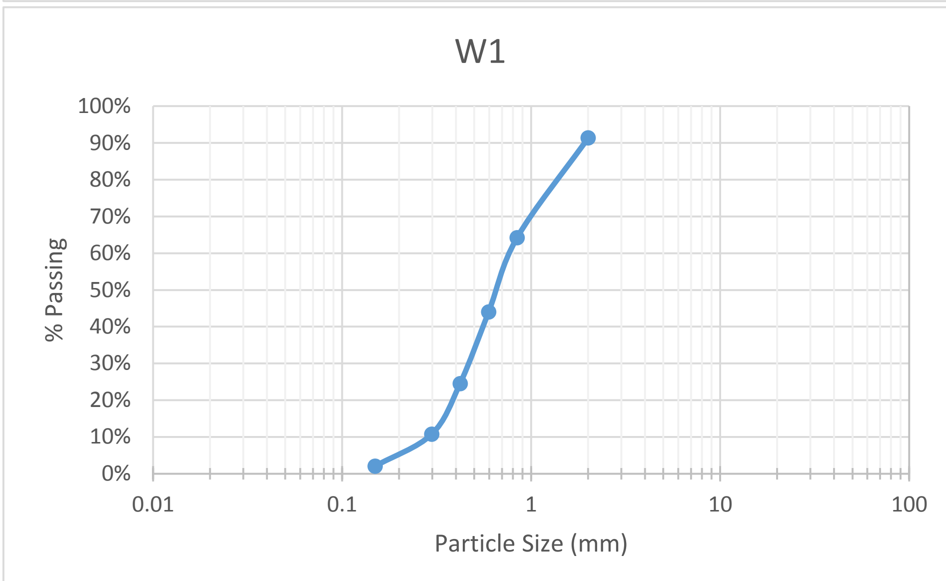
1
10
100
Particle Size (mm)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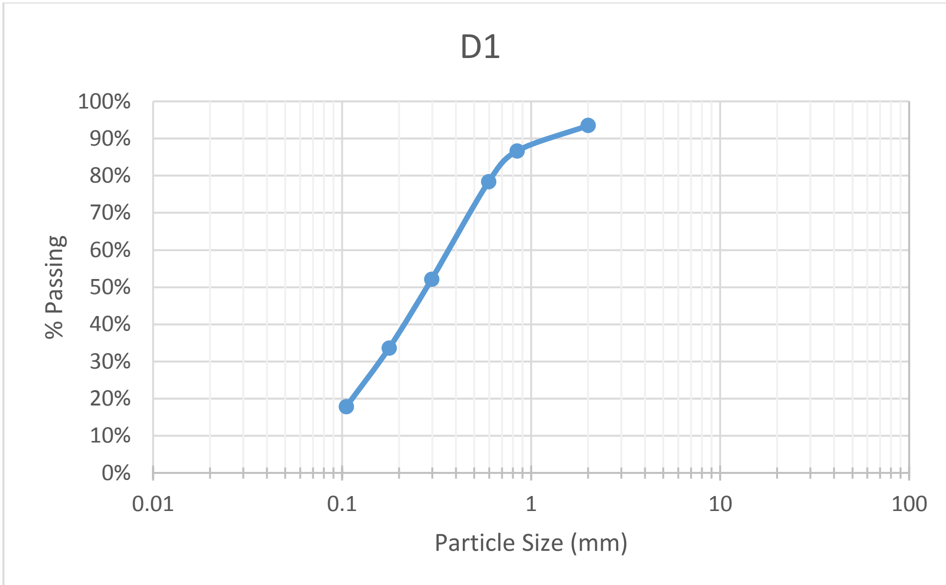
80%
70%
60%
50%
40%
30%
20%
10%
0%
% Passing
0.01
0.1
1
10
100
Particle Size (mm)



D1
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
% Passing
0.01
0.1
1
10
100
Particle Size (mm)

Appendix E

MATLAB Code for Manning’s Equation

```
close all;


figure;
plot(num,elev,'-*','linewidth',2)
grid on;
title('Typical Cross Section')
hold on
xlabel('Horizontal width, ft.')
ylabel('Elevation, ft.')
%mannings equation
A = 20;%ft^3
P = 16.65; %ft

d_et = 0.002; % 1.12mm
d_et_in = 0.0393701*d_et; %inches
Rh = A/P; %feet
Rhm = 0.3048*Rh; %meters

S = 0.01; %Slope ft/ft
n = ((0.8204).*Rhm.^(1/6))./(1.16 + 2.0.*log(Rhm./d_et)); %Mannings
%Using SI units for Rhm, Dimensionless

figure;
plot(d_et,n,'linewidth',2)
grid on
xlabel('Diameter of the particle')
ylabel('n, roughness coefficient')

Q = (1.49./n)*A.*Rh^(2/3)*sqrt(S) %Channel Flow Flowrate
 figure;
 plot(d_et,Q,'linewidth',2)
grid on;
 xlabel('Diameter, d_{80} (mm)')
 ylabel('Flowrate, Q(cfs)')
```

Appendix F

BMP Major Design Elements

Table F-1. Major Design Elements of Stormwater Wetland (NCDENR Stormwater BMP Manual)

Required by the NC Administrative Rules of the Environmental Management Commission. Other specifications may be necessary to meet the stated pollutant removal requirements.	
1	Sizing shall take into account all runoff at ultimate build-out including off-site drainage.
2	Side slopes stabilized with vegetation shall be no steeper than 3:1.
3	Wetland shall be located in a recorded drainage easement with a recorded access easement to a public ROW.
4	The wetland must drawdown in 2-5 days.
5	Flow through the wetland shall not be short-circuited and shall be made as lengthy as possible.
6	A forebay is required.
7	A vegetated filter strip is not required for overflows or discharges from a constructed stormwater wetland (except for within ½ mile of and draining to SA waters or unnamed tributaries of SA waters). For SA waters in Phase II areas, criteria in S.L. 2006-246 Section 9(h) must still be met.
Required by DWQ policy. These are based on available research, and represent what DWQ considers necessary to achieve the stated removal efficiencies.	
8	Sizing of the wetland is based on storage volume requirements as described in this section.
9	The minimum treatment volume for a stormwater wetland shall be 3,630 ft^3. Lesser volumes will be approved on a case-by-case basis.
10	Maximum shallow land depth (temporary pool) shall be one (1) foot.
11	Minimum length to width ratio shall be 1.5:1, however, 3:1 is preferred.
12	The wetland must be stabilized within 14 days of construction.
13	The PPE must be maintained by installation of a natural or synthetic liner with a maximum infiltration rate of 0.01 in/hr beneath the entire bottom of deep pools and shallow water areas to prevent excessive seepage unless shallow groundwater is present and the PPE is located within 6-inches of the SHWT. In this case, only lining of deep pools may be required. At least 4-inches of appropriate topsoil must be added to the liner to support plant growth.
14	Cattails are not to be planted.

Table F-2. Major Design Elements of Wet Extended Detention Basin (NCDENR Stormwater BMP Manual)

Required by the NC Administrative Rules of the Environmental Management Commission. Other specifications may be necessary to meet the stated pollutant removal requirements.	
1	Sizing shall take into account all runoff at ultimate build-out, including off-site drainage.
2	Vegetated slopes shall be no steeper than 3:1.
3	BMP shall be located in a recorded drainage easement with a recorded access easement to a public ROW.
4	Basin discharge shall be evenly distributed across a minimum 30 feet long vegetative filter strip unless it is designed to remove 90% TSS. (A 50-ft filter is required in some locations.)
5	If any portion is used for S&EC during construction must be cleaned out and returned to design state.
6	The design storage shall be above the permanent pool.
7	Discharge rate of the treatment volume shall completely draw down between 2 and 5 days.
8	The average depth of the permanent pool shall be a minimum of 3 feet. The average depth shall be calculated as described in Figure 10-2b.
9	Permanent pool surface area shall be determined using Tables 10-1, 10-2, 10-3, and 10-4.
10	The flow within the pond shall not short-circuit the pond.
11	BMP shall be designed with a forebay.
12	Basin side slopes shall be stabilized with vegetation above the permanent pool level.
13	The pond shall be designed with side slopes below the 10ft shelf stabilized per what the soils will support and per the PE's judgment.
14	The basin shall be designed with sufficient sediment storage to allow for proper operation between scheduled cleanouts.
Required by DWQ policy. These are based on available research, and represent what DWQ considers necessary to achieve the stated removal efficiencies.	
15	BMP shall not be located to produce adverse impacts on water levels in adjacent wetlands.
16	A minimum 10-foot wide vegetated shelf shall be installed around the perimeter. The inside edge of the shelf shall be 6″ below the permanent pool elevation; the outside edge of the shelf shall be 6″ above the permanent pool elevation.
17	The forebay volume should be about 20% of the total permanent pool volume, leaving about 80% of the design volume in the main pool.
18	Freeboard shall be a minimum of 1 foot above the maximum stage of the basin.
19	The permanent pool elevation shall be within 6 inches (plus or minus) of the SHWT elevation.

Table F-3. Major Design Elements of Dry Extended Detention Basin (NCDENR Stormwater BMP Manual)

Required by the NC Administrative Rules of the Environmental Management Commission. Other specifications may be necessary to meet the stated pollutant removal requirements.	
1	Sizing shall take into account all runoff at ultimate build-out including off-site drainage.
2	Vegetated side slopes shall be no steeper than 3:1.
3	BMP shall be located in a recorded drainage easement with a recorded access easement to a public ROW.
4	If the BMP is used for sedimentation and erosion control during construction, it must be cleaned out and returned to the design state.
5	For pollutant removal credit, the applicable design storm must be held for a period of no less than 2 but no more than 5 days.
6	BMP shall have an additional 25% storage volume for sediment deposition.
Required by DWQ policy. These are based on available research, and represent what DWQ considers necessary to achieve the stated removal efficiencies.	
7	Seasonally high groundwater table must be at least 2 feet below the bottom of the basin.
8	The energy of the influent flow must be controlled.
9	The maximum depth shall be 10 feet.
10	Freeboard shall be a minimum of 1 foot above the maximum stage of the basin.
11	A minimum legnth to width ratio of 3:1 is recommended. A minimum length to width ratio of 1.5:1 is required.
12	A sediment depth indicator must be provided.
13	Basin design must include a drain.
14	A forebay is required if the design volume is over 10 acre-inches.

Appendix G

NOAA Precipitation Data

NOAA Atlas 14, Volume 2, Version 3
Location name: Cashiers, North Carolina, US*
Latitude: 35.1134°, Longitude: -83.0991°
Elevation: 3488 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**0.439** (0.399-0.484)	**0.524** (0.476-0.577)	**0.615** (0.559-0.678)	**0.695** (0.628-0.764)	**0.791** (0.711-0.870)	**0.870** (0.777-0.961)	**0.949** (0.841-1.05)	**1.03** (0.903-1.14)	**1.13** (0.982-1.27)	**1.22** (1.05-1.38)
10-min	**0.701** (0.638-0.773)	**0.838** (0.761-0.923)	**0.986** (0.894-1.08)	**1.11** (1.00-1.22)	**1.26** (1.13-1.39)	**1.39** (1.24-1.53)	**1.51** (1.34-1.67)	**1.63** (1.43-1.81)	**1.79** (1.55-2.01)	**1.93** (1.65-2.17)
15-min	**0.876** (0.797-0.966)	**1.05** (0.957-1.16)	**1.25** (1.13-1.37)	**1.41** (1.27-1.54)	**1.60** (1.44-1.76)	**1.75** (1.57-1.94)	**1.91** (1.69-2.11)	**2.06** (1.81-2.29)	**2.25** (1.95-2.52)	**2.42** (2.07-2.73)
30-min	**1.20** (1.09-1.32)	**1.46** (1.32-1.60)	**1.77** (1.61-1.95)	**2.04** (1.84-2.24)	**2.37** (2.13-2.60)	**2.64** (2.36-2.92)	**2.92** (2.59-3.23)	**3.20** (2.81-3.56)	**3.58** (3.11-4.02)	**3.91** (3.35-4.42)
60-min	**1.50** (1.36-1.65)	**1.82** (1.66-2.01)	**2.27** (2.06-2.50)	**2.65** (2.40-2.92)	**3.15** (2.83-3.47)	**3.58** (3.20-3.95)	**4.02** (3.56-4.45)	**4.49** (3.94-5.00)	**5.14** (4.46-5.76)	**5.71** (4.89-6.46)
2-hr	**1.76** (1.61-1.95)	**2.14** (1.95-2.36)	**2.65** (2.41-2.92)	**3.10** (2.81-3.41)	**3.70** (3.33-4.07)	**4.22** (3.77-4.66)	**4.77** (4.22-5.28)	**5.36** (4.70-5.95)	**6.21** (5.37-6.94)	**6.97** (5.95-7.85)
3-hr	**1.97** (1.80-2.18)	**2.39** (2.18-2.63)	**2.94** (2.68-3.25)	**3.45** (3.13-3.79)	**4.15** (3.73-4.57)	**4.77** (4.26-5.26)	**5.43** (4.80-6.01)	**6.16** (5.39-6.85)	**7.24** (6.23-8.10)	**8.21** (6.97-9.28)
6-hr	**2.73** (2.51-3.01)	**3.28** (3.01-3.61)	**3.99** (3.65-4.39)	**4.64** (4.23-5.11)	**5.57** (5.03-6.13)	**6.40** (5.73-7.06)	**7.30** (6.46-8.09)	**8.31** (7.27-9.25)	**9.82** (8.41-11.0)	**11.2** (9.44-12.7)
12-hr	**3.80** (3.49-4.16)	**4.56** (4.19-4.99)	**5.54** (5.08-6.06)	**6.39** (5.84-7.00)	**7.55** (6.86-8.29)	**8.56** (7.71-9.42)	**9.62** (8.59-10.6)	**10.8** (9.52-11.9)	**12.5** (10.8-13.9)	**14.0** (12.0-15.7)
24-hr	**4.93** (4.57-5.36)	**5.92** (5.48-6.42)	**7.28** (6.73-7.91)	**8.38** (7.72-9.10)	**9.90** (9.07-10.7)	**11.1** (10.2-12.1)	**12.5** (11.3-13.6)	**13.8** (12.4-15.1)	**15.8** (14.0-17.4)	**17.5** (15.4-19.3)
2-day	**6.04** (5.60-6.53)	**7.22** (6.70-7.81)	**8.80** (8.14-9.52)	**10.1** (9.29-10.9)	**11.8** (10.9-12.8)	**13.2** (12.1-14.4)	**14.7** (13.4-16.0)	**16.3** (14.7-17.8)	**18.5** (16.5-20.4)	**20.4** (18.0-22.6)
3-day	**6.50** (6.05-7.00)	**7.75** (7.21-8.35)	**9.36** (8.70-10.1)	**10.6** (9.87-11.5)	**12.4** (11.5-13.4)	**13.8** (12.7-14.9)	**15.3** (14.0-16.6)	**16.8** (15.3-18.3)	**18.9** (17.0-20.7)	**20.7** (18.4-22.8)
4-day	**6.97** (6.51-7.48)	**8.28** (7.74-8.90)	**9.93** (9.26-10.7)	**11.2** (10.5-12.1)	**13.0** (12.0-14.0)	**14.4** (13.3-15.5)	**15.8** (14.5-17.1)	**17.3** (15.8-18.7)	**19.3** (17.5-21.1)	**21.0** (18.9-23.1)
7-day	**8.32** (7.76-8.96)	**9.88** (9.21-10.6)	**11.8** (11.0-12.8)	**13.4** (12.5-14.5)	**15.6** (14.4-16.8)	**17.3** (15.9-18.6)	**19.1** (17.5-20.6)	**20.9** (19.0-22.6)	**23.5** (21.2-25.5)	**25.5** (22.9-27.9)
10-day	**9.63** (8.99-10.3)	**11.4** (10.6-12.3)	**13.5** (12.6-14.5)	**15.2** (14.2-16.4)	**17.5** (16.3-18.9)	**19.4** (17.9-20.9)	**21.3** (19.6-22.9)	**23.2** (21.2-25.0)	**25.9** (23.5-28.0)	**28.0** (25.2-30.5)
20-day	**12.9** (12.1-13.7)	**15.1** (14.3-16.1)	**17.6** (16.6-18.7)	**19.5** (18.3-20.7)	**21.9** (20.6-23.3)	**23.8** (22.3-25.3)	**25.6** (23.9-27.3)	**27.4** (25.5-29.3)	**29.7** (27.5-31.9)	**31.5** (29.0-33.9)
30-day	**15.6** (14.8-16.5)	**18.3** (17.3-19.4)	**21.0** (19.8-22.2)	**23.0** (21.7-24.3)	**25.5** (24.1-27.0)	**27.4** (25.8-29.0)	**29.1** (27.4-30.9)	**30.8** (28.9-32.8)	**33.0** (30.8-35.2)	**34.6** (32.1-37.0)
45-day	**19.8** (18.8-20.8)	**23.1** (22.0-24.3)	**26.1** (24.8-27.4)	**28.2** (26.8-29.7)	**30.8** (29.3-32.4)	**32.7** (31.0-34.4)	**34.4** (32.5-36.2)	**35.9** (33.9-37.9)	**37.8** (35.6-40.1)	**39.2** (36.8-41.6)
60-day	**23.7** (22.6-24.8)	**27.6** (26.3-28.9)	**30.8** (29.5-32.3)	**33.2** (31.7-34.8)	**36.0** (34.3-37.7)	**37.9** (36.1-39.8)	**39.7** (37.7-41.7)	**41.3** (39.2-43.4)	**43.2** (40.9-45.6)	**44.5** (42.1-47.1)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).
Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.
Please refer to NOAA Atlas 14 document for more information.

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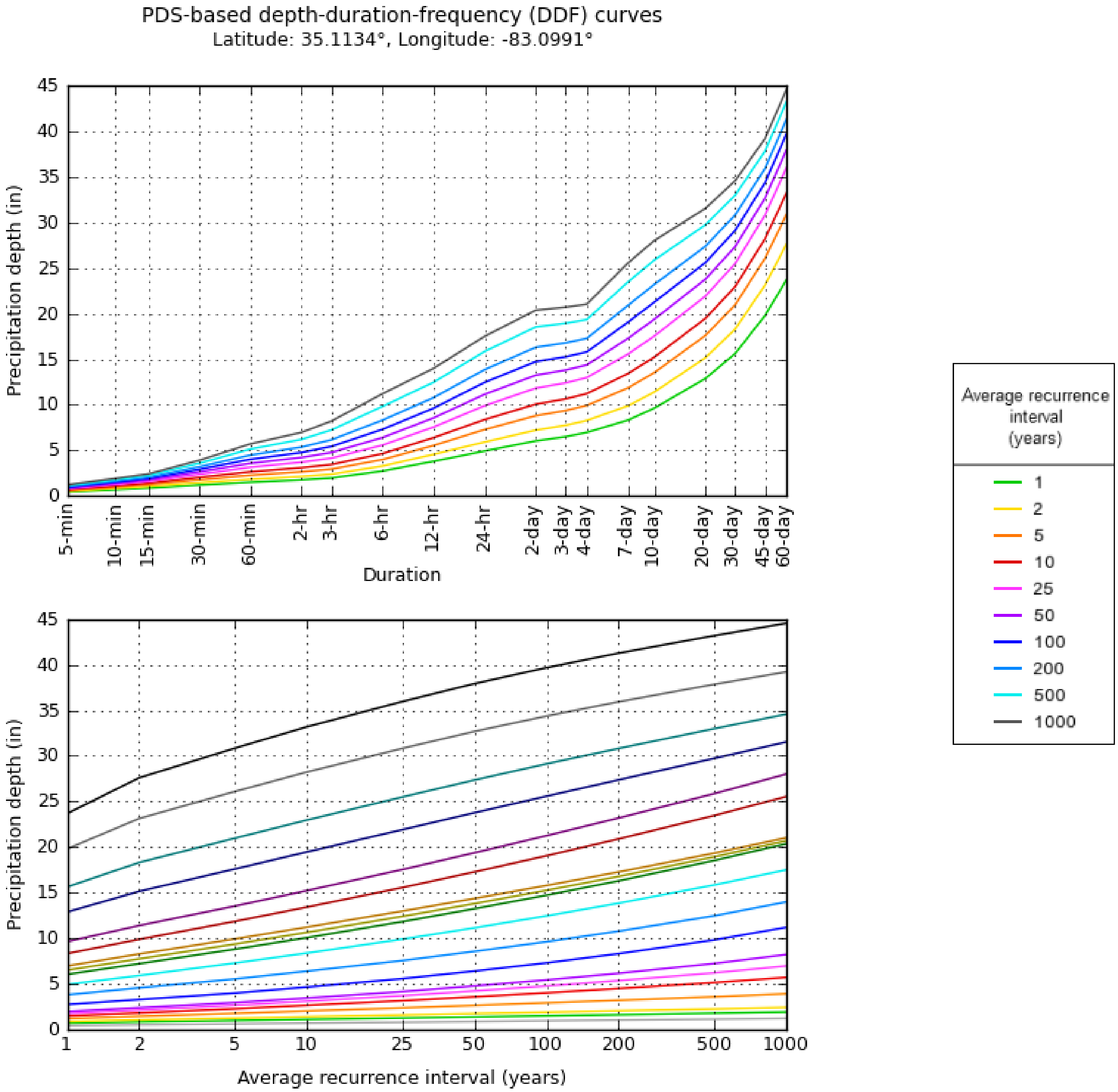
PF graphical



NOAA Atlas 14, Volume 2, Version 3

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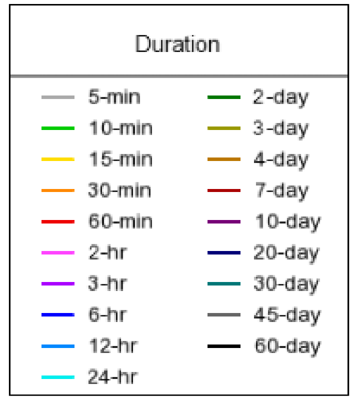

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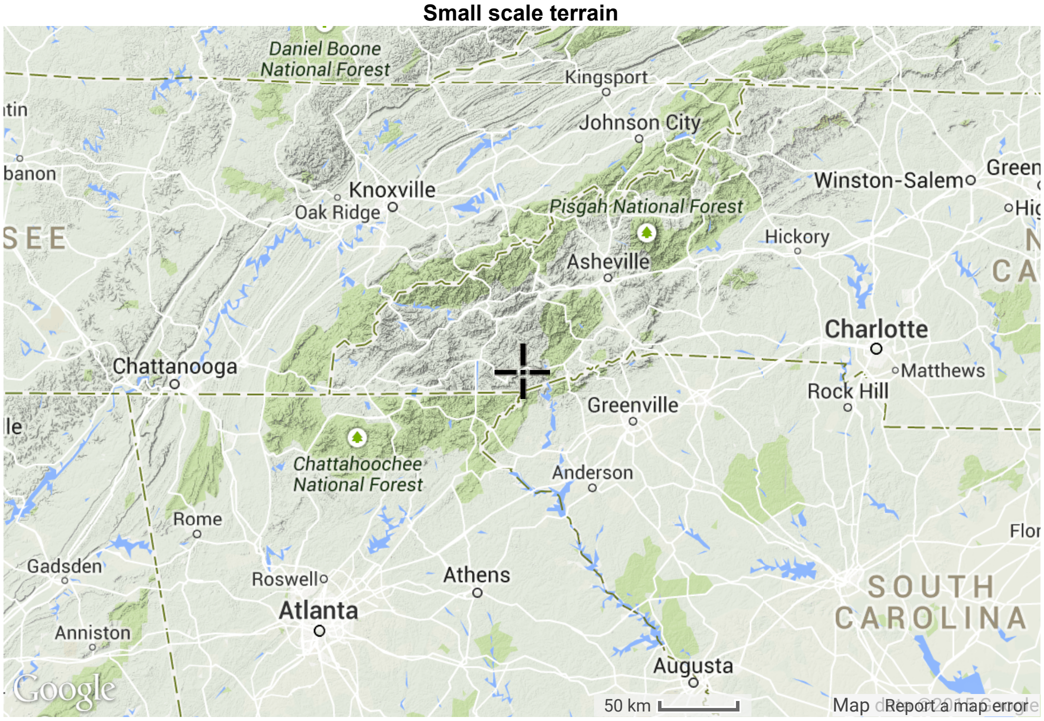
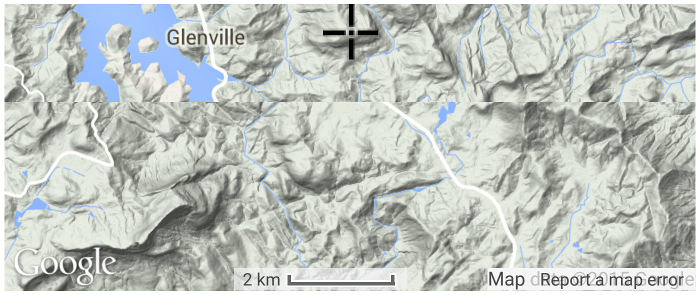
Maps & aerials

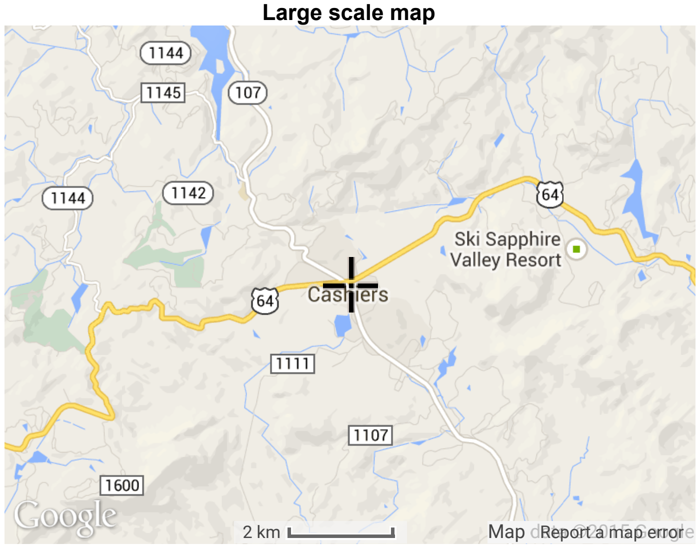
Small scale terrain



Large scale terrain



Large scale map



Large scale aerial

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Appendix H

Detention Basin Storage Volume Calculations

Detention Pond Volume Estimation - U.S. units
Option 1 - Basin near Frank Allen

1. The Generalized Model

Inputs

Watershed area, **A** =	480	acres
Peak Runoff Rate (before), q_{pb} =	33	cfs
Peak Runoff Rate (after), q_{pa} =	56	cfs
Time of Concentration (before), t_{cb} =	0.9	hr
Time of Concentration (after), t_{ca} =	0.78	hr

Calculations

Runoff Depth - after, Q_a =	0.090	in
Peak Runoff Ratio, α =	0.59	
Time ratio, γ =	1.15	
$(2 - \gamma)/\alpha$ =	1.44	
for $(2 - \gamma)/\alpha > 1$, V_s =	0.033	in
Vol. in acre ft, V_{st} =	1.334	acre-ft
for $(2 - \gamma)/\alpha \leq 1$, V_s =	0.029	in
Vol. in acre ft, V_{st} =	1.169	acre-ft

$\alpha = q_{pb}/q_{pa}$

$\gamma = t_{cb}/t_{ca}$

$$V_s = Q_a\left(\frac{\gamma + \alpha + \alpha\gamma(\gamma + \alpha - 4)}{\gamma - \alpha}\right)$$

for $\alpha < 2 - \gamma$

$$V_s = Q_a\left(\frac{\gamma - \alpha}{\gamma + \alpha}\right)$$

for $\alpha \geq 2 - \gamma$

Generalized Method Equations

$V_{st} = V_s A/12$ acre-ft

(V_s in inches and A in acres gives V_{st} in acre-ft)

For either 'before' or 'after' conditions:

$Q = (120/121)(q_p\, t_c/A)$

(Q in inches, q_p in cfs, t_c in hr, A in acres)

Detention Pond Volume Estimation - U.S. units
Option 2 - Basin near US-64

1. The Generalized Model

Inputs

Watershed area, **A** =	480	acres
Peak Runoff Rate (before), $\mathbf{q_{pb}}$ =	33	cfs
Peak Runoff Rate (after), $\mathbf{q_{pa}}$ =	60	cfs
Time of Concentration (before), $\mathbf{t_{cb}}$ =	0.62	hr
Time of Concentration (after), $\mathbf{t_{ca}}$ =	0.72	hr

Calculations

Runoff Depth - after, $\mathbf{Q_a}$ =	0.089	in
Peak Runoff Ratio, α =	0.55	
Time ratio, γ =	0.86	
(2 - γ)/α =	2.07	
for **(2 - γ)/α > 1**, $\mathbf{V_s}$ =	0.053	in
Vol. in acre ft, $\mathbf{V_{st}}$ =	2.123	acre-ft
for **(2 - γ)/α $\leq$ 1**, $\mathbf{V_s}$ =	0.020	in
Vol. in acre ft, $\mathbf{V_{st}}$ =	0.787	acre-ft

$$V_s = Q_a\left(\frac{\gamma + \alpha + \alpha\gamma(\gamma + \alpha - 4)}{\gamma - \alpha}\right)$$

$$\text{for} \quad \alpha < 2 - \gamma$$

$$V_s = Q_a\left(\frac{\gamma - \alpha}{\gamma + \alpha}\right)$$

$$\text{for} \quad \alpha \geq 2 - \gamma$$

Generalized Method Equations

$\alpha = q_{pb}/q_{pa}$

$\gamma = t_{cb}/t_{ca}$

$V_{st} = V_s A/12$ acre-ft

(V_s in inches and A in acres gives V_{st} in acre-ft)

For either 'before' or 'after' conditions:

$Q = (120/121)(q_p\, t_c/A)$

(Q in inches, q_p in cfs, t_c in hr, A in acres)

Detention Pond Volume Estimation - U.S. units
Option 1 - Basin near Frank Allen

1. The Generalized Model

Inputs

Watershed area, **A** = 480 acres

Peak Runoff Rate (before), $\mathbf{q_{pb}}$ = 18 cfs

Peak Runoff Rate (after), $\mathbf{q_{pa}}$ = 33 cfs

Time of Concentration (before), $\mathbf{t_{cb}}$ = 0.91 hr

Time of Concentration (after), $\mathbf{t_{ca}}$ = 0.72 hr

$$V_s = Q_a\left(\frac{\gamma + \alpha + \alpha\gamma(\gamma + \alpha - 4)}{\gamma - \alpha}\right)$$

for $\alpha < 2 - \gamma$

$$V_s = Q_a\left(\frac{\gamma - \alpha}{\gamma + \alpha}\right)$$

for $\alpha \geq 2 - \gamma$

Generalized Method Equations

Calculations

Runoff Depth - after, $\mathbf{Q_a}$ = 0.049 in

Peak Runoff Ratio, α = 0.55

Time ratio, γ = 1.26

(2 - γ)/α = 1.35

for **(2 - γ)/α > 1**, $\mathbf{V_s}$ = 0.020 in

Vol. in acre ft, $\mathbf{V_{st}}$ = 0.818 acre-ft

for **(2 - γ)/α ≤ 1**, $\mathbf{V_s}$ = 0.019 in

Vol. in acre ft, $\mathbf{V_{st}}$ = 0.780 acre-ft

$\alpha = q_{pb}/q_{pa}$

$\gamma = t_{cb}/t_{ca}$

$V_{st} = V_s A/12$ acre-ft

(V_s in inches and A in acres gives V_{st} in acre-ft)

For either 'before' or 'after' conditions:

$Q = (120/121)(q_p t_c/A)$

(Q in inches, q_p in cfs, t_c in hr, A in acres)

Appendix I

ASHTO Value of Time Calculations

Monetary Value of Travel Time Calculations Using AAHSTO Standards (Page 1 of 2)

USDOT provided guidelines and procedures for calculating the value of travel time lost by road users.
The monetary value of travel time is based on the concept that time spent traveling otherwise would have been spent productively, whether for remunerative work or recreation
Transportation Benefit-Cost Analysis

Total estimated delay time per day

Traffic Vol.	Total Delay Time (min/veh)	Delay Time for All Vehicles (veh-hours/day)
200	5	16.666667

Monetary Value of Personal Travel Time

Step 1) Proportion of passenger cars on personal travel

2009 NHTS	Personal	Business
	93.7	6.3

Step 2) Average vehicle occupancy of passenger cars

2009 NHTS	Personal Travel - Vacation
	2.7

Step 3) Estimate per hour monetary value of travel time for a person on personal travel

Cashiers Median Household Income [Source]
42,699

Hourly value of personal travel time per person is calculated as:
For local personal travel,
Hrly val. of personal travel time/person = 50% of median annual household income / 2080 hours

Hourly value of personal travel time per person =	10.264183 /person-hr

Step 4) Compute per hour monetary value of travel time for a vehicle on personal travel
For local personal travel,
Hourly value of travel time for a vehicle on personal travel = Step 2 * Step 3

Hourly value of travel time for a vehicle on personal travel =	27.713293 /vehicle-hr

Step 5) Compute travel delay costs for passenger cars on personal travel
Total delay time for passenger cars on personal travel
= Average delay time * Number of passenger car vehicles on persontal travel
Total delay costs for passenger cars on personal travel
= Total delay time for passenger cars on personal travel * hourly $ value of vehicle delay time

Total delay time for passenger cars on personal travel = 15.616667 veh-hrs/day
Total delay costs for passenger cars on personal travel = 432.78926 $/day

Monetary Value of Business Travel Time

Step 1) Determine the proportion of passenger cars on business travel

2009 NHTS	Personal	Business
	93.7	6.3

Step 2) Average vehicle occupancy of passenger cars

2009 NHTS Business
1.24

Continued on next page

Monetary Value of Travel Time Calculations Using AAHSTO Standards (Page 2 of 2)

Step 3) Estimate per hour monetary value of travel time for a person on business travel
Hourly value of a person's time on business travel = 100% of median hourly wages plus benefits
Hourly employment cost (Dec, 2014) = 33.13 http://www.bls.gov/news.release/ecec.t01.htm
Hourly employment cost (Mar, 2014) = 33.32878 http://www.bls.gov/news.release/eci.t01.htm

Step 4) Compute per hour monetary value of travel time for a vehicle on business travel
Hourly time value of a vehicle on business travel
= hourly value of a person's time on business travel * average vehicle occupancy
Hourly time value of a vehicle on business travel = 41.327687 /veh-hr

Step 5) Compute travel delay costs for passenger cars on business travel
Total delay time for passenger cars on business travel
= Average delay time * Number of passenger car vehicles on business travel
Total delay costs for passenger cars on business travel
= Total delay time for passenger cars on business travel * hourly $ value of vehicle delay time

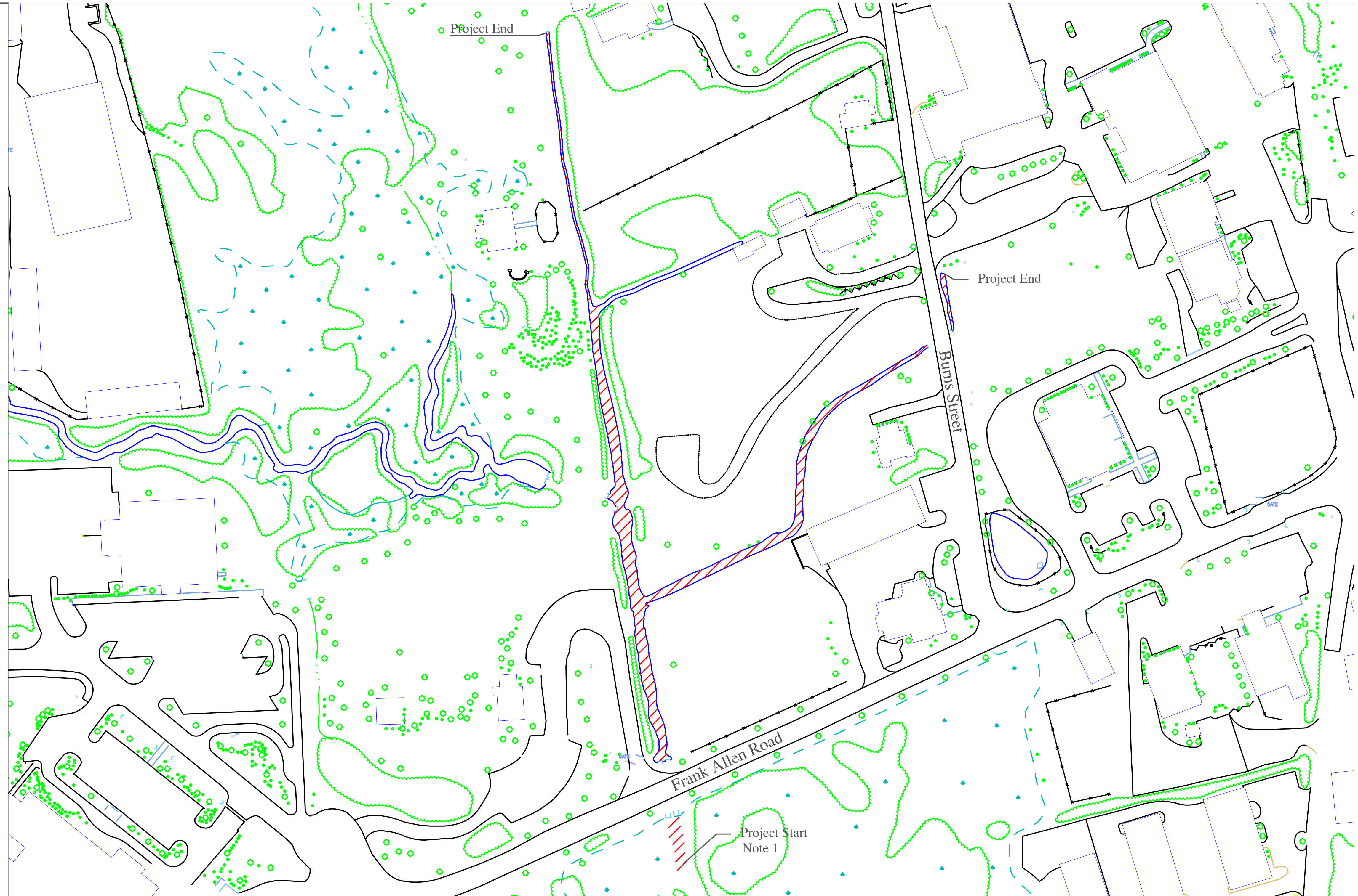
Total delay time for passenger cars on business travel = 1.05 veh-hrs/day
Total delay costs for passenger cars on business travel = 43.394072 $/day

Appendix J

Sediment Removal Project Area

Notes

1. Cashiers Creek was not mapped south of Frank Allen Road, but sediment removal should occur there. Survey of the area is required to determine how far south sediment removal can occur.



Sediment Removal Project Area

Trees

Marsh

Fence

Culvert

Sediment Removal Project Area

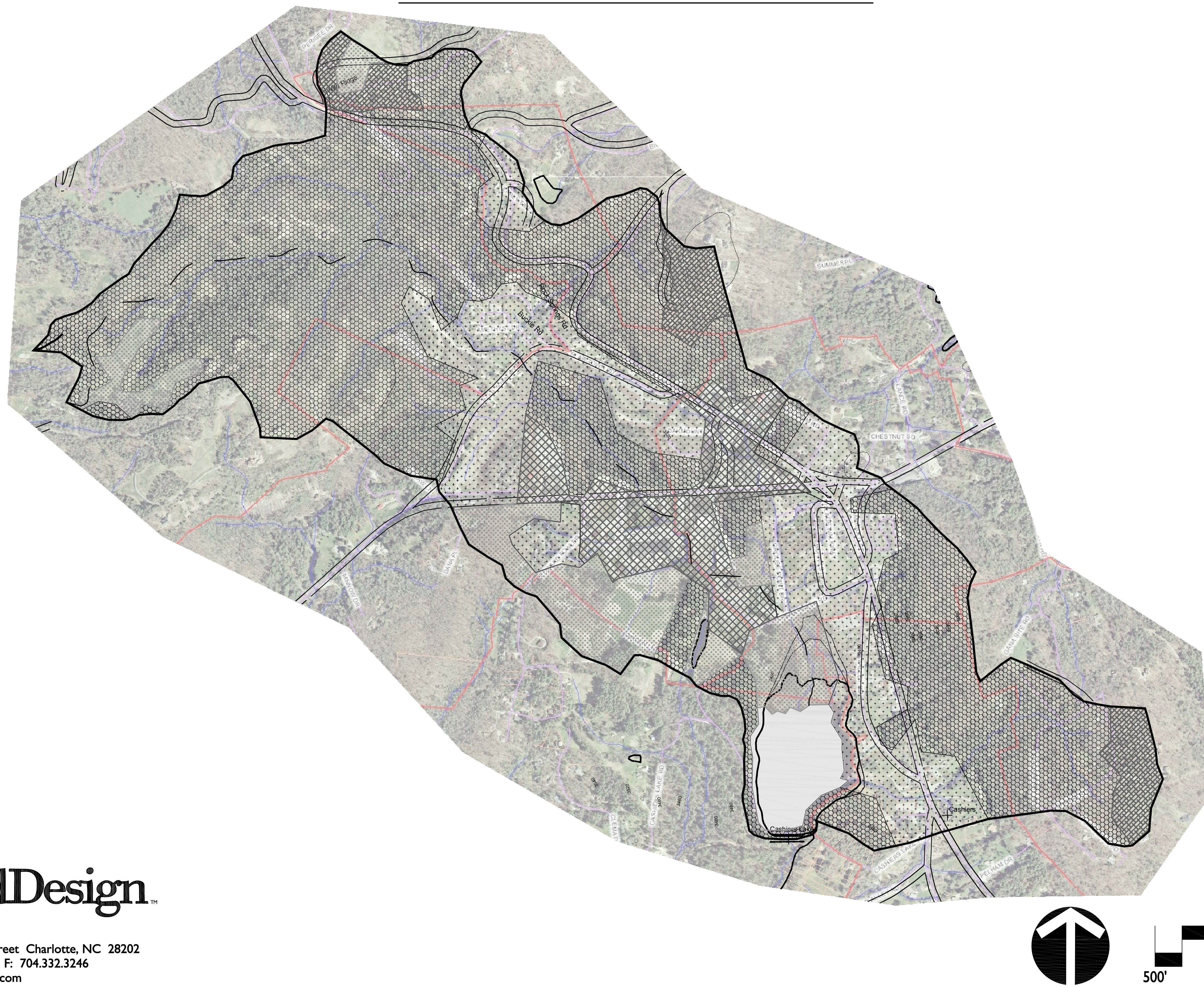
RENOVATUS SOLUTIONS

more than engineers

Appendix K

Existing Land Use Map

EXISTING LAND USE MAP



LEGEND

RESIDENTIAL-1 ACRE LOTS

RESIDENTIAL-2 ACRE LOTS

COMMERCIAL & BUSINESS

OPEN SPACE

WOODS

WATER- CASHIERS LAKE

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