FINAL REPORT
BASINWIDE DRAINAGE SYSTEM ANALYSIS
AND
WATER SURFACE PROFILE DETERMINATION

TOWN OF CARY
WAKE COUNTY, NORTH CAROLINA

Prepared by:
Dewberry & Davis, Inc.
Raleigh, North Carolina

August 8, 2006

Dewberry

Town of Cary Basinwide Drainage System Analysis and Water Surface Profile Determination

Engineering Seal

I hereby state that this Town of Cary Basinwide Drainage System Analysis and Water Surface Profile Determination Study Report was prepared by me or under my direct supervision.

Signed, sealed, and dated this 23rd day of August, 2006.

By:

Kenneth W. Ashe, P.E., CFM
Project Manager – Water Resources
Dewberry & Davis, Inc.
2301 Rexwoods Drive, Suite 200
Raleigh, North Carolina 27607
919-881-9939
NC PE No. 027462

NORTH CAROLINA PROFESSIONAL SEAL 027462 ENGINEER KENNETH W. ASHE
8/23/06

Dewberry
Dewberry & Davis, Inc.
2301 Rexwoods Drive, Suite 200
Raleigh, NC 27607-3366
Phone: 919.881.9939
Fax: 919.881.9923
www.dewberry.com

EXECUTIVE SUMMARY

To address flooding concerns and assess costs of potential capital improvement projects associated with flooding in the downtown Town Center Area Plan (TCAP), the Town of Cary commissioned the Basinwide Drainage System Analysis and Water Surface Profile Determination study. This study was developed with three primary goals. These included development of water surface profiles for the larger drainage area streams, development of hydraulic grade lines for local channel and pipe systems and providing master plan solutions for identified problem areas.

Goal one focused on development of multiple recurrence interval water surface profiles for Walnut Creek, Coles Branch, and Swift Creek Tributary 7 from the limit of the existing Federal Emergency Management Agency (FEMA) and/or the North Carolina Floodplain Mapping Program (NCFMP) study limits upstream to the point the drainage area of the stream is approximately 50 acres. This analysis was conducted using the standard FEMA study storms and provided base flood elevations for approximately three (3) miles of open channel.

The second goal was to develop existing and future conditions hydraulic grade lines for open and closed drainage systems with drainage areas of approximately 10 acres within the Town's long range Town Center Area Plan (TCAP). This analysis modeled open channel reaches and pipe systems concurrently for three (3) storm events, the typical "summer storm", the 2-year storm, and the 10-year storm which is used for pipe size design within the Town.

Each of the first two goals provided the base data needed for the third goal which was to develop the stormwater master planning presented in this Final Report, which considers existing and anticipated future problem areas, conceptual design solutions for the identified problems, and preliminary planning level costs for the identified solutions.

The TCAP area is 2,286 acres of mixed urban and residential development. The existing stormwater conveyance system serving the TCAP area includes approximately 26 miles of pipe (12 inches or greater in size) and 17.2 miles of channel (based on the Town's Pilot and Maynard Loop stormwater inventory collected between 2002 and 2003). This project focuses on 2.9 miles of piped network and 7.2 miles of open channel modeling, which is essentially the primary drainage conveyance for the TCAP area.

The analysis was completed utilizing previous modeling and mapping from FEMA, NCFMP, the City of Raleigh, and Town of Cary, as available. A combination of field survey and Town of Cary digital data, including topographic information, planimetrics, ortho-photography, and stormwater inventory were used as the basis for the modeling.

The open and closed system hydraulic analysis of the stormwater infrastructure (hydraulic grade line analysis) was performed on fifteen individual systems for three storm events, as directed by the Town, with a specific focus on a simulation of a short summer storm event and the Town's pipe design standard of a 10-year recurrence interval storm event. The Town's desired level of service was established to be two separate, but related requirements in the 10-year storm event: 1) there would be no structure or street flooding; and 2) the hydraulic grade line would be contained below the crown of the pipe. The results from the existing and future conditions analyses reveal that significant portions of the existing drainage system in the TCAP area do not meet the desired level of service or current design standards. Numerous areas show the potential for roadway, property, and structure flooding, as well as other problems such as erosive velocities as a result of the undersized system.

Results of the water surface profile analysis and hydraulic grade line (HGL) open and closed system study were compared with the Town's planimetrics for identification of potential structure flooding. One

hundred forty six (146) buildings were identified from the results of the large drainage open channel modeling during the water surface profile analysis and eighty two (82) buildings were identified from the open and closed system modeling during the hydraulic grade line analysis to be at risk of flooding, either from surcharging pipes or open channel flooding.

Several considerations for potential improvements, such as localized infrastructure improvements, extension or relocation of closed systems, and channel stabilization were considered to reduce these flooding problems. Based on discussions with the Town, eleven of the fifteen systems studied in the HGL analysis were analyzed for potential master planning improvements. Three of the systems not studied had flood risks associated with private systems, and the remaining system was not studied because the HGL analyses did not indicate flood risks for the studied storm events. A single alternative for each area studied was chosen as the best alternative for the Town to consider implementing (See Table 1), based on the cost and feasibility of each alternative. Master planning was not conduced on the reaches of Walnut Creek, Coles Branch and Swift Creek Tributary 7 studied as part of the water surface profile determination.

The recommended alternatives for each system were analyzed for overage prioritization using a prioritization matrix. This approach, assessing benefit and cost relationships between selected alternatives allows weighting of factors to identify a ranking of improvements for implementation. Organizing the recommended projects will assist the Town in prioritizing when the projects can be completed based on available funding. The main considerations that were used to create the prioritization list include:

- Number of roads with a level of service being improved to greater than the 10-year future storm event
- Level of Utility Conflict
- Easement Cost
- Permitting Difficulty
- Number of structures with a level of service being improved to greater than the 10-year future storm event
- Cost of the Project

The prioritization list along with the estimated probable construction cost and future cost index is shown in Table 1.

Table 1 Prioritization Preliminary Planning Level Construction Cost to Complete Recommended Alternatives

Priority	Area	Chosen Alternative	Cost					
			2006	2007	2008	2009	2010	2011
1	10	Alternative 2	$604,000	$625,140	$647,020	$669,666	$693,104	$717,363
2	9	Alternative 2	$282,000	$291,870	$302,085	$312,658	$323,601	$334,928
3	15	Alternative 1	$208,000	$215,280	$222,815	$230,613	$238,685	$247,039
4	8	Alternative 2	$1,678,000	$1,736,730	$1,797,516	$1,860,429	$1,925,544	$1,992,938
5	12	Alternative 1	$107,000	$110,745	$114,621	$118,633	$122,785	$127,082
6	4	Alternative 1	$647,000	$669,645	$693,083	$717,340	$742,447	$768,433
7	5	Alternative 1	$167,000	$172,845	$178,895	$185,156	$191,636	$198,344
8	2	Alternative 1	$65,000	$67,275	$69,630	$72,067	$74,589	$77,200
9	3	Alternative 1	$685,000	$708,975	$733,789	$759,472	$786,053	$813,565
10	13	Alternative 1	$867,000	$897,345	$928,752	$961,258	$994,902	$1,029,724
11	6	Alternative 1	$773,000	$800,055	$828,057	$857,039	$887,035	$918,082
Total Cost to Complete			$6,083,000	$6,295,905	$6,516,263	$6,744,331	$6,980,381	$7,224,698

From the prioritization matrix process it was determined that Area 10 will provide the greatest potential reduction in flood risk to structures and roads for the cost, level of permitting required, and degree of utility interference encountered. The current total estimated probable construction costs for the selected alternatives described in this report are $6,083,000.

After the water surface profile analysis almost all of the crossings analyzed did not meet a 100-year level of service desired by the Town of Cary. Subsequent open channel analysis was performed to evaluate the cost of upgrading culverts in the studied areas to provide a 100-year level of service. Analysis revealed that while most crossings could be upgraded to provide a 100-year level of service with increased conveyance area, that not all crossing could attain the desired level of service without significant channel and culvert improvements. Table 2 provides planning level construction costs to complete the recommended alternatives that would upgrade the majority of the crossings studied in this report to 100-year level of service.

Table 2 Planning Level Construction Cost to Achieve 100 year Level of Service for Roadways in Water Surface Profile Analysis

Area Description	100 Year Event Cost					
	2006	2007	2008	2009	2010	2011
Coles Branch	$ 218,145	$ 225,780	$ 233,682	$ 241,861	$ 250,326	$ 259,087
Swift	$4,445,568	$4,601,163	$4,762,204	$4,928,881	$ 5,101,392	$ 5,279,940
Walnut	$4,182,217	$4,328,594	$4,480,095	$4,636,899	$ 4,799,190	$ 4,967,162
Total Cost	$8,845,929	$9,155,537	$9,475,981	$9,807,640	$10,150,908	$10,506,189

The cost of this study and the recommend capital improvement projects represent less than 15% of the reported tax value of the identified structures at risk of flooding, so the cost of the recommended improvements are small when compared to the overall value of property that is targeted for reduction in overall flood risk. Based on this assumed benefit to cost and the number and value of structures determined to be at risk of flooding in the TCAP, the Town should consider a number of recommended actions to help alleviate flood risks. Based on the value of potential development, the Town should regulate development using the 100-year water surface elevations developed herein, or the Town should consider including the 100-year floodplains on the FEMA DFRIMs. To determine the flood risk on other open channel reaches, similar studies to 100-year water surface elevations should be conducted throughout the Town limits and ETJ. Additionally, it is recommended that the Town consider changes to the existing development ordinances to require new development to include assessments of impacts to downstream stormwater infrastructure as part of the permitting requirements and to require the development to alleviate any problem areas that are created.

Table of Contents

EXECUTIVE SUMMARY ..... **II**
ACRONYMS ..... **VIII**
1.0 Introduction **1**
2.0 Hydrologic Modeling **3**
2.1 HEC-HMS Model 3
2.1.1 Subbasin Delineation 4
2.1.2 Hydrologic Soil Groups 4
2.1.3 Land Use 4
2.1.4 Curve Number Calculation 5
2.1.5 Lag Time 5
2.1.6 Locations of Attenuation 6
2.2 Summer Storm Determination 7
2.3 Discharge Calculations 8
2.4 Impervious Surfaces 9
2.5 Quality Control 9
2.6 Basin Hydrology Notes 10
3.0 Water Surface Profile Analysis and Floodplain Mapping **11**
3.1 Modeling Approach 11
3.2 Hydraulic Data and Model Development 11
3.3 Peak Discharges 12
3.4 Reach Boundary Conditions 12
3.5 Cross Sections 12
3.6 Manning's n Values 13
3.7 Stream Crossings 13
3.8 Ineffective Flow Areas 14
3.9 Expansion and Contraction Coefficients 15
3.10 Summary of Hydraulic Results 15
3.11 Quality Control of Hydraulic Modeling and Mapping 16
3.12 Discussion of Hydraulic Results 17
3.13 Roadway Level of Service Analysis Summary of Results 17
3.13.1 Swift Creek Tributary 7 17
3.13.2 Coles Branch 20
3.13.3 Walnut Creek 22
3.14 Roadway Level of Service Analysis Discussion of Results 24
4.0 Hydraulic Grade Line Analysis **25**
4.1 Modeling Approach 25
4.2 Hydraulic Data and Model Development 26
4.3 Peak Discharges 26
4.4 Reach Boundary Conditions 27
4.5 Cross Sections 27
4.6 Manning's n Values 28
4.7 Culverts 28
4.8 Gutters 28
4.9 Summary of Hydraulic Results 28

4.9.1 Area 1: Walnut Creek Basin 29
4.9.2 Area 2: Walnut Creek Basin 29
4.9.3 Area 3: Walnut Creek Basin 29
4.9.4 Area 4: Walnut Creek Basin 30
4.9.5 Area 5: Walnut Creek Basin 31
4.9.6 Area 6: Swift Creek Basin 31
4.9.7 Area 7: Swift Creek Basin 32
4.9.8 Area 8: Swift Creek Basin 32
4.9.9 Area 9: Swift Creek Basin 32
4.9.10 Area 10: Coles Branch Basin 33
4.9.11 Area 11: Coles Branch Basin 33
4.9.12 Area 12: Black Creek Basin 34
4.9.13 Area 13: Black Creek Basin 34
4.9.14 Area 14: Black Creek Basin 35
4.9.15 Area 15: Black Creek Basin 36
4.10 Additional Modeling Notes 37
4.11 Quality Control 40
5.0 Conceptual Design Level Alternatives 41
5.1 General Approach 41
5.1.1 Development of Alternatives 41
5.1.2 Cost Estimation of Alternatives 42
5.1.3 Utility Conflicts 43
5.2 Alternatives Analysis and Recommendations 43
5.2.1 Area 1 – No Master Planning (per the Town) 43
5.2.2 Area 7 – No Master Planning (per the Town) 43
5.2.3 Area 11 – No Master Planning (per the Town) 43
5.2.4 Area 14 – No Master Planning (per the Town) 43
5.2.5 Area 2 – Master Planning Alternatives and Recommendation 44
5.2.6 Area 3 – Master Planning Alternatives and Recommendation 46
5.2.7 Area 4 – Master Planning Alternatives and Recommendation 50
5.2.8 Area 5 – Master Planning Alternatives and Recommendation 55
5.2.9 Area 6 – Master Planning Alternatives and Recommendation 57
5.2.10 Area 8 – Master Planning Alternatives and Recommendation 59
5.2.11 Area 9 – Master Planning Alternatives and Recommendation 63
5.2.12 Area 10 – Master Planning Alternatives and Recommendation 68
5.2.13 Area 12 – Master Planning Alternatives and Recommendation 72
5.2.14 Area 13 – Master Planning Alternatives and Recommendation 75
5.2.15 Area 15 – Master Planning Alternatives and Recommendation 78
5.3 Recommendation Summary and Project Prioritization 80
6.0 Discussion of Results and Recommended Future Activities 84
6.1 Extended Discussion of Results 84
6.2 Recommended Future Activities 84
Appendix A – Location Maps ..... - 1 -
Appendix B - Hydrology Basin Maps ..... - 1 -
Appendix C - Hydrology Summary Data ..... - 1 -
Appendix C - 1 – Land Use Summaries by Basin ..... - 2 -

Appendix C - 2 – Existing CN Values and Impervious Area Summary ..... - 6 -
Appendix C - 3 – Future CN Values and Impervious Area Summary ..... - 8 -
Appendix C - 4 – Existing and Future Curve Numbers by Basin..... - 11 -
Appendix C - 5 – Time of Concentration Calculation Summary ..... - 12 -
Appendix C - 6 – Time of Concentration Flow Paths ..... - 14 -
Appendix C - 7 – Stage Storage Routing Information for WC5 Attentuation..... - 18 -
Appendix C - 8 – 50-Acre Calculated Flows..... - 19 -
Appendix C - 9 – 10-Acre Calculated Flows..... - 23 -
Appendix C - 10 – Quality Control for "Large Scale Hydrology"..... - 32 -
Appendix D - Hydraulic Supplemental Data and Results ..... - 1 -
Appendix D - 1 – Peak Discharges Used in Open Channel Modeling ..... - 2 -
Appendix D - 2 – Boundary Conditions for Water Surface Profile Determination..... - 3 -
Appendix D - 3 – Computed Water Surface Elevations from HEC-RAS ..... - 4 -
Appendix D - 4 – Floodplain Mapping..... - 9 -
Appendix E - Hydraulic Grade Line Supporting Data ..... - 1 -
Appendix E - 1 – Reach Boundary Conditions..... - 2 -
Appendix F - HGL Results, Level of Service Tables, and Area Maps..... - 1 -
Appendix F - 1 – Area 1 ..... - 1 -
Appendix F - 2 – Area 2 ..... - 1 -
Appendix F - 3 – Area 3 ..... - 1 -
Appendix F - 4 – Area 4 ..... - 1 -
Appendix F - 5 – Area 5 ..... - 1 -
Appendix F - 6 – Area 6 ..... - 1 -
Appendix F - 7 – Area 7 ..... - 1 -
Appendix F - 8 – Area 8 ..... - 1 -
Appendix F - 9 – Area 9 ..... - 1 -
Appendix F - 10 – Area 10 ..... - 1 -
Appendix F - 11 – Area 11 ..... - 1 -
Appendix F - 12 – Area 12 ..... - 1 -
Appendix F - 13 – Area 13 ..... - 1 -
Appendix F - 14 – Area 14 ..... - 1 -
Appendix F - 15 – Area 15 ..... - 1 -
Appendix G – HGL Profiles..... - 1 -
Appendix G - 1 – Area 1..... - 1 -
Appendix G - 2 – Area 2..... - 1 -
Appendix G - 3 – Area 3..... - 1 -
Appendix G - 4 – Area 4..... - 1 -
Appendix G - 5 – Area 5..... - 1 -
Appendix G - 6 – Area 6..... - 1 -
Appendix G - 7 – Area 7..... - 1 -
Appendix G - 8 – Area 8..... - 1 -
Appendix G - 9 – Area 9..... - 1 -
Appendix G - 10 – Area 10..... - 1 -
Appendix G - 11 – Area 11..... - 1 -
Appendix G - 12 – Area 12..... - 1 -
Appendix G - 13 – Area 13..... - 1 -

Appendix G - 14 – Area 14 ..... - 1 -
Appendix G - 15 – Area 15 ..... - 1 -

Table 1 Prioritization Preliminary Planning Level Construction Cost to Complete Recommended Alternatives III
Table 2 Planning Level Construction Cost to Achieve 100 year Level of Service for Roadways in Water Surface Profile Analysis II
Table 3 Flow Percent Increase from Existing to Future Land Use Conditions 5
Table 4 Study Streams 11
Table 5 Limits of Detail Study 12
Table 6 Manning's n Ranges 13
Table 7 Crossings modeled in HEC-RAS 14
Table 8 Number of Floodprone Structures Per Basin 16
Table 9 Level of Service Table 16
Table 10 Level of Service Table for Swift Creek Tributary 7 (100 yr) 18
Table 11 Detailed Level of Service Results for Swift Creek Watershed 19
Table 12 Level of Service Table for Coles Branch (100 yr) 20
Table 13 Detailed Level of Service Results for Swift Creek Watershed 21
Table 14 Level of Service Table for Walnut Creek (100 yr) 22
Table 15 Detailed Level of Service Results for Walnut Creek Watershed 23
Table 16 Cost Estimate Summary for Level of Service Improvements 24
Table 17 Hydraulic Grade Line Analysis Study Areas 25
Table 18 Summary of Area Master Plan Response 41
Table 19 Criterion Evaluation for Selected Alternatives 82
Table 20 Selected Alternative Priority Ranking Matrix 82
Table 21 Indexed Costs for Each Selected Alternative 83

ACRONYMS

FEMA – Federal Emergency Management Agency
NCFMP – North Carolina Floodplain Mapping Program
TCAP – Town's Center Area Plan
HGL – Hydraulic Grade Line
WSE – Water Surface Elevation
HEC-HMS – Hydrologic Engineering Centers Hydrologic Modeling System
HEC-RAS – Hydrologic Engineering Centers River Analysis System
T_c – Time of Concentration
CN – Curve Number
SCS – Soil Conservation Service
TR – Technical Release
DWM – Dynamic Watershed Model
NRCS – Natural Resources Conservation Service
NWS – National Weather Service
TIN – Triangular Irregular Network
GIS – Global Information System
GPS – Global Positioning System
NEH – National Engineering Handbook
AMC – Antecedent Moisture Content
RCP – Reinforced Concrete Pipe
CMP – Corrugated Metal Pipe
USACE – United States Army Corps of Engineers
CIP – Capital Improvement Project
EPCC – Estimates of Probable Construction Cost
FFE – Finished Floor Elevation
NFIP – National Flood Insurance Program

1.0 Introduction

This report generally follows the same outline as the project work flow, summarized as follows:

- Hydrology
- Water Surface Profile Determination and Floodplain Mapping
- Hydraulic Grade Line Analysis on Open/Closed Stormwater Systems in the TCAP Area
- Master Planning of Improvements to Systems with Flooding Problems
- Preliminary Planning Level Cost Estimates, Recommendations, and Prioritization of Recommended Master Planning Alternatives
- Summaries, Maps, and Results including:
  - Location Maps
  - Hydrology Basin Maps
  - Hydrology Summary Data
  - Hydraulic Supplemental Data and Results
  - Hydraulic Grade Line Supporting Data
  - Hydraulic Grade Line Results, Level of Service Tables, and Area Maps
  - Hydraulic Grade Line Profiles

This report relates the results of an engineering planning study performed for the Town of Cary to analyze the existing drainage systems within the Town Center Area Plan (TCAP) vicinity. The primary focus of the study was to determine flood risks, potential causes of flooding, and recommend alternative solutions. The TCAP is a long-range master plan for Town of Cary's downtown, which will provide a basis for future development in the downtown area. As a part of the master planning effort the Town determined the need to analyze the existing drainage systems to identify level of service deficiencies in the system based on the current land use as well as future deficiencies caused by changes in land use molded by the proposed Land Use Plan that was adopted by the Cary Town Council in November of 1996.

The study consisted of information review and compilation, field data collection, hydrologic and hydraulic analysis of open and closed drainage systems, identification of potential drainage system problem areas, and determination of improvements to reduce potential flood risks. This report represents the completion of nine (9) work elements associated with this study, which are:

1. Project Management
2. Data Collection / GIS Manipulation
3. Hydrologic Modeling
4. Water Surface Profile Analyses
5. Hydraulic Grade Line Analyses
6. Floodplain Mapping
7. Master Plan
8. Final Report
9. Supplemental Options

A hydrologic analysis was conducted to calculate peak surface runoff flows and to assess the general hydrologic response of the watershed for a range of rainfall events and different land use conditions. The analysis was conducted for the 2-, 10-, 25-, 50-, and 100-year design storm events along with the 10-year/1-hour storm event using SCS methodology with the Hydrologic Engineering Centers Hydrologic Modeling System (HEC-HMS) rainfall-runoff model. These flows were used directly in the open channel hydraulics modeling and indirectly for the hydraulic grade line analysis, which required further distinction of flows to determine inflow into the individual stormwater systems.

Hydraulic analysis included creating existing and future floodplains and hydraulic grade lines for areas within the TCAP vicinity. The results of the hydraulic analysis were used to provide suggestions to

reduce existing and future flood risks. Multiple recurrence interval water surface profiles were determined for Walnut Creek, Walnut Creek Tributary, Coles Branch and Swift Creek Tributary 7 from the limit of existing Federal Emergency Management Agency (FEMA) and/or the North Carolina Floodplain Mapping Program (NCFMP) study limits to the point the drainage area of the stream is approximately 50 acres. Existing and future conditions hydraulic grade lines were analyzed for open and closed stormwater systems with drainage areas of approximately 10 acres or larger within the TCAP area. The results of the water surface and hydraulic grade line profile analysis were used to determine potential flood risks in the areas studied.

Potential flood risks were analyzed to determine reasonable, cost-effective improvements to the systems that reduce the flood risks. This portion of the study involved considerations of how the existing systems are constructed and how the systems could be improved. First, the limitations of improving the systems were determined such as, the systems relative location to structures, roadways, and infrastructure, and also the available depth of cover for pipes and structures based on upstream and downstream constraints. Knowing the constraints that exist in the systems provided a useful tool in producing improvements that avoided or minimized conflicts in the ability to design a particular alternative. Preliminary planning level costs were associated with each feasible design, and cost saving alternatives were utilized when possible.

2.0 Hydrologic Modeling

A hydrologic analysis was conducted to calculate peak surface runoff flows and to assess the general hydrologic response of each watershed for a range of rainfall events and different land use conditions. Hydrology was generated in two (2) separate tasks, "large scale hydrology" and "small scale hydrology". The "large scale hydrology" developed the discharges used in the water surface profile determination and formed the base data used to develop the "small scale hydrology" utilized for the HGL analyses.

The hydrologic analyses were conducted for the 2-, 10-, 25-, 50-, and 100-year design storm events along with the 10-year/1-hour storm event using the Soil Conservation Services (SCS) Technical Release 55 (TR-55, 1986) methodology within the HEC-HMS rainfall-runoff model. Basin characteristics such as land use, soils, topography, basin and subbasin delineation, time of concentration, and curve numbers were developed for the analysis. Discharges derived from HEC-HMS, utilizing GIS support data, for the 50-acre hydrology ("large scale hydrology") were used in the water surface profile determination.

The results of the "large scale hydrology" were used as the basis to create the "small scale hydrology", for the 2- and 10-year design storms and the 10-year/1-hour storm event. The Town's stormwater inventory data, land use data, and planimetrics (utilized to create an impervious coverage) were used to calculate area-weighted discharges at locations within the stormwater infrastructure with 10 or more acres of contributing drainage. The locations of pipe size changes were reviewed for identification of potential significant influxes of contributing flow and/or potential problem areas based on decreasing pipe sizes or lack of pipe size change. A further degree of urbanization is anticipated in the future conditions in the TCAP area according to the future land use classifications. Therefore, future runoff potential was also modeled for the watersheds to anticipate the results of the reduction in ability to allow infiltration and a decrease the travel time of runoff.

This section is organized in the general order of analysis followed during discharge determination. Section 2.1 provides information for inputs used in the HEC-HMS model;: Section 2.2 gives a general description of development, including use of GIS data, of the "large scale" and "small scale" discharges; Section 2.3 describes how impervious percentages were calculated; and Section 2.4 gives a general description and results of quality control efforts.

2.1 HEC-HMS Model

A systematic approach to creating the peak flows for use in the hydraulic models was applied based on the SCS TR-55 methodology due to the level of detail required for this analysis. Each of the four watersheds within the TCAP area being studied was delineated into approximate 50-acre basins. The soil type and land use that is found in each basin was used to determine the precipitation losses associated with each basin, which is referred to as the curve number (CN) of the basin. The time of concentration (T_c) for each basin was then determined using TR-55 methodology, which was used to determine the lag time for each basin. Precipitation depths were taken from a depth-duration-frequency curve frequently used with the TR-55 method created by the Natural Resource Conservation Service (NRCS) using data from the National Weather Service (NWS) TR-40.

The four main inputs that are required for HEC-HMS to determine the peak flow of a sub-basin are the basin area, curve number, lag time, and the storms precipitation depth.

2.1.1 Subbasin Delineation

The TCAP boundary encompasses four major watersheds; namely, Coles Branch, Swift Creek, Walnut Creek and Black Creek. Each watershed was sub-divided into 50-acre basins to be used in the "large scale hydrology" for modeling existing streams (See Appendix B). 50-acre basins were chosen to be consistent with the upstream limits of study for the water surface profile analysis.

To delineate the target 50-acre size subbasins, a GIS enabled program called the Dynamic Watershed Model (DWM) was used. DWM is a flexible set of tools in GIS environment that allow a user to do much of the pre-processing work needed to do hydrologic and hydraulic modeling associated with urban watersheds, including delineating basins, calculating CN values, integrating survey data, and other useful processes. DWMs automated terrain processing routines were used to convert the Town of Cary's 2-foot contour topography shapefile into a triangular irregular network (TIN) of the study boundary, which was then converted to Grid data. The next sequence of automated procedures consisted of "burning in" streams/pipes, filling sinks, calculating flow direction and flow accumulation, creating streams and stream links, creating outlets and finally delineating subwatersheds. This step enforces flow connectivity based on the hydrography and pipe network into the GIS flow network.

Following creation of the 50-acre basins, these basins were used as a reference to develop the more detailed 10-acre subbasins. The 10-acre subbasins were delineated with basin breaks for major points of interest, such as road crossings and confluences using the same automated routines and manually adding outlets at specific points of interest. To add further detail, the 10-acrea basins for the entire stormwater system, including the high priority areas designated by the Town, were manually modified to account for pipe system flow collection at each inlet structure within the 10-acre subbasins (inlet drainage areas). The "small scale hydrology" basins can also be viewed in Appendix B.

2.1.2 Hydrologic Soil Groups

Hydrologic soil groups are necessary for quantifying the amount of water that can infiltrate into the soil to reduce the quantity of runoff in a particular watershed. There are four hydrologic soil groups, A – D, each representing different infiltration characteristics. To determine the hydrologic soils present in the TCAP area, a raw soil shapefile was downloaded from Wake County's GIS website at http://www.wakegov.com/tax/propertyandmapping/gisdigitaldata.htm. The hydrologic soil group information which was absent from the raw county soils shapefile was manually attributed by matching the soil map symbol and the corresponding hydrologic soil group information from the Natural Resources Conservation Service's published soil survey information which can be downloaded at http://soils.usda.gov/survey/printed_surveys/north_carolina.html. By analysis of the downloaded data, it was determined that 92 percent of the soils in the TCAP area are hydrologic soil group B, seven (7) percent are hydrologic soil group C, and the rest of the soils were hydrologic group D.

2.1.3 Land Use

Land use classifications generally show the degree to which a particular parcel is undeveloped or built-out based on a land use type. The TCAP, approved by Cary Town Council in November of 1996, involves changing future land use, so discharge determinations for the existing and future land use conditions were used as a basis of determination of where future problem areas will arise and determination of where existing areas have problems. Land use information was provided by the Town of Cary as separate GIS layers for the existing and future conditions. The existing land use shapefile was processed to add roads as a separate classification and thus has thirteen (13) unique land use classifications. The future land use shapefile is similar to the existing land use shapefile but with more land use categories. The future land

use categories are based on classification for planning purposes rather than major changes in the land use. The future land use shapefile did not contain data in the north western corner of the study area; hence the existing land use classifications were appended to the future land use in the void area and the resulting shapefile was processed to include roads as unique land use category leading to a total of seventeen (17) unique land use categories in the future land use shape file.

In general commercial, residential, and institutional area increased, and vacant land is completely used up in the future land use conditions (Appendix C - 1). The impacts of these changes in land use can be seen in Table 3, which illustrates that with the use of the remainder of vacant land along with changes in land use overall, the flows in each basin are anticipated to increase. Basins that have more vacant land in the existing conditions typically have a greater flow increase in the future conditions, since this land will be converted into commercial, residential, or institutional land in the future.

Table 3 Flow Percent Increase from Existing to Future Land Use Conditions

		2yr	10yr	25yr	50yr	100yr	10yr 1hr
Walnut Creek	Existing Flows (7% Vacant)	707	1419	1858	2204	2549	614
	Future Flows (0% Vacant)	809	1541	1986	2334	2681	760
	Percent Increase	**14**	**9**	**7**	**6**	**5**	**24**
Swift Creek	Existing Flows (11% Vacant)	929	1908	2516	2997	3478	794
	Future Flows (0% Vacant)	1141	2186	2815	3311	3805	1095
	Percent Increase	**23**	**15**	**12**	**10**	**9**	**38**
Coles Branch	Existing Flows (14% Vacant)	535	1088	1427	1696	1967	460
	Future Flows (0% Vacant)	671	1257	1608	1885	2156	641
	Percent Increase	**25**	**16**	**13**	**11**	**10**	**39**
Black Creek	Existing Flows (15% Vacant)	581	1202	1586	1891	2198	476
	Future Flows (0% Vacant)	704	1360	1757	2071	2381	625
	Percent Increase	**21**	**13**	**11**	**10**	**8**	**31**

2.1.4 Curve Number Calculation

The CN values were determined based on information compiled about the soils and land use, and results in a value that represents the precipitation losses associated with each basin. Curve numbers were assigned to each land use-soil group condition using information from the existing FEMA NCFPM data for Wake County, published SCS information, and sampled data with CNs calculated based on percent impervious layer. An existing and future CN look up table can be viewed in Appendix C - 2 and Appendix C - 3, respectively.

After development of the CN lookup tables, the next step was to calculate weighted average CNs for each subbasin in Swift Creek, Walnut Creek, Coles Branch and Black Creek watersheds. The existing and future condition land use, soils, and the subbasins were spatially intersected in GIS to obtain polygons representing every unique combination of land use and soils within each subbasin. CNs based on AMC-II conditions were assigned to each polygon using the CN lookup tables described in the paragraph above. Finally, a composite CN was calculated for each subbasin by computing the area-weighted average of the individual CN polygons within the subbasin, which can be viewed in Appendix C - 4.

2.1.5 Lag Time

To determine lag time, the T_c value for a particular basin must be determined, after which a simplistic conversion factor is multiplied by the T_c to determine the lag time. Lag time is typically referred to as the

difference in time from the centroid of the rainfall excess (the water that is available for runoff after interception, infiltration, and depression storage have been accounted for) to the peak of the hydrograph, or peak flow of the storm, which is commonly calculated by multiplying the T_c by 0.6. The T_cs for this project were calculated using the methods described in TR-55 (SCS, 1986). This method entails subdividing the T_c flow path (i.e. flow path from the hydraulically farthest point to the outlet) into distinct flow regimes/segments, calculating travel times for each segment based on physical characteristics, and then summing up the individual segment travel times to obtain a T_c for a given drainage area. TR-55 divides the T_c flow path into the following flow regimes:

Sheet Flow – flow over plan surfaces occurring in the headwater of a given drainage area
Shallow Concentrated Flow – concentrated flow through rivulets, gutters, and other shallow features
Open Channel Flow – flow through defined open channels
Pond – flow through ponds and detention areas

TR-55 provides methods and equations for calculating sheet, shallow concentrated, and open channel flow travel times. Closed and open channel flow travel times were determined using FlowMaster by assuming full flow in pipes and a reasonable depth for open channels depending on the side slope and bottom width. Average slopes for pipes and channels and channel geometry information were verified in GIS. For cases with no inventory data, reasonable values based on topography and results for similar TCAP cases were assumed.

T_cs were developed for each subbasin using aerial images, contours, stormwater inventory, and other relevant information. T_c flow paths were drawn in GIS and subdivided into the different flow regime segments. Several T_c paths were generally assumed and tested to determine the greatest T_c. Length and slope for each segment were calculated in GIS. Sheet flow segments were restricted to 100 feet or less in length, per the TR-55 methodology. Values of 3.6 inches and (0.03-0.08) were used for the 2-year, 24-hour rainfall (P2) and the overland Manning n value (N) sheet flow input parameters, respectively. Velocities for open channel segments were calculated in FlowMaster assuming Manning's equation, bank-full conditions and using channel geometries derived from field surveyed open system inventory data. Velocities through pipes were determined in FlowMaster assuming Manning's equation, full flow conditions and using field surveyed stormwater inventory data. A velocity of 1ft/sec was assumed for flow across small detention basins in the study area if they were not determined to require stage-storage discharge calculations. Travel time for each individual flow segment was calculated using equations/parameters specified in TR-55. The T_c for each subbasin was then calculated by summing the travel times of the individual T_c flow path segments. Finally, T_c values for each basin were multiplied by 0.6 to convert them to lag times.

The same T_c values were used for both the existing and future conditions as a simplified assumption, that each watershed is nearly fully developed and that the main drainage patterns were not expected to change significantly. These T_c values for each basin can be viewed in Appendix C - 5, and the T_c flow paths can be viewed in Appendix C - 6.

2.1.6 Locations of Attenuation

Attenuation is the reduction of flow caused by intermittent storage in a basin, including sumps, storage behind culverts, detention basins, etc. Attenuation is an important factor in determining the final discharge in a basin, as an attenuation area can reduce flow significantly. Attenuation in the Cary basin-wide drainage system was accounted for through the use of normal depth based hydrologic routing reaches and level-pool storage elements in the hydrologic model. Normal depth routing cross sections

were drawn across routing reaches and 8-point cross section takeoffs were entered into HEC-HMS for open system sections. In addition, level-pool storage routing was used in Walnut Creek subbasin WC5 to reflect the potential for flood waters to back-up behind the embankment along SE Maynard. Potential attenuation was not considered to be significant for routing reaches consisting of closed pipe systems. In these reaches, the input hydrographs were simply translated (i.e. shift hydrograph in time without attenuation) using the hydrograph lag method. All other ponds in the study area are small, low level ponds that are not anticipated to reduce flow significantly and therefore ignored for routing.

Elevation-Area routing parameters used for level-pool routing in the storage element (See Appendix C - 7) were developed from 2-foot contour topographic information provided by the Town.

2.2 Summer Storm Determination

Discussions with Town staff during the scoping and preliminary phase of the project indicated the need to model a storm event that emulated summer flood events. To determine the storm event, an analysis of hydrologic response of the TCAP watersheds was conducted using the several assumptions, namely, the scope of work was developed for design storms and did not include hydrograph development, the focus of the "small scale hydrology" was the response of pipe systems, and the scale of the "small scale hydrology" is consistent with use of the rational method.

Gage data acquired from the NC State Climate Office for the Raleigh area between 1948 and May 2005 was reviewed and it was determined that summer storm events generally occur in durations ranging between 1 and 8 to 12 hours. Discussions with Town staff indicated flooding events occur on a fairly frequent basis, so the smallest statistically valid recurrence interval, the 2-year storm, was added to the analysis. To ensure capture of the critical storm, three recurrence intervals, the 2-, 5- and 10-year storms, were used with the 1-, 2-, 4-, 8-, and 12-hour storm durations.

Seven (7) representative TCAP watersheds, with varying land use and drainage areas, were selected for the analysis. The rational method discharges for each of the storm events noted above were determined for each watershed based on the watershed drainage, flow path slope, and land use. To determine the overall hydrologic response of the watershed the discharges were routed through the pipe or culvert which forms the outlet for the watershed, to determine which storm event created the greatest potential for flooding at the outlet pipe/culvert.

The analysis indicated the 5-year 1-hour storm and the 10-year 1-hour storm events created the greatest potential flooding at the watershed outlets. Both events produced similar peak discharges, so in effort to be conservative, the 10-year 1-hour storm event was chosen, based on the larger total rainfall volume associated with the event. The results and selected design were discussed and approved by the Town prior to final implementation of the storm event in the hydrologic analysis.

As part of the quality control described in Section 2.4 herein, the selected storm event was compared to the general information contained in the gage data for the Raleigh area as supplied by the NC State Climate Office. The 10-year / 1-hour storm has a total rainfall of 2.3 inches. The entire gage data was reviewed, along with summer data (June through October) for period between 1980 and 2005 (the period covering the majority of the Town's growth) for two (2) rainfall totals, daily rainfall that exceeded 2 inches and hourly rainfall that exceeded 2 inches. The gage date indicated two (2) inch daily rainfall totals occurred more frequently than once a year for the period of record and more frequently than once a summer for the period between 1980 and 2005. Additionally, the data indicated two (2) inch hourly events occurred, on average, once every 6 years and once every 8 summers. Three (3) inch daily totals do appear in the gage record, however, there were no hourly rainfall totals that exceeded three (3) inches.

Based on the data provided by the NC State Climate Office for the Raleigh area, it appears that use of a 1-hour storm event with total rainfall of 2.32 inches is consistent with the peak hourly rainfall experienced both annually and during summer months. Storm events greater than two (2) inches in an hour occur at least once a summer, and the selected storm has a larger total rainfall than rain events that occur more frequently.

2.3 Discharge Calculations

Different levels of detail in the flows were required when analyzing the open channels and the open/closed systems, so a "large scale hydrology" method and a "small scale hydrology" method were used.

The "large scale hydrology" is associated with the delineated 50-acre basins, which were used to determine the initial peak flows for each basin based on the TR-55 methodology. The open channel systems require less detailed basin delineations for determining flows, since during open channel modeling, flows associated with a particular basin are used from the top of that basin to the bottom of that basin.

The "small scale hydrology" is associated with basins with a target size of 10-acres that are subdivided from the 50-acre basin. The "small scale hydrology" is calculated indirectly from the flows calculated in the "large scale hydrology" utilizing a ratio of the impervious area of the target 10-acre basins. "Small scale hydrology" uses a more detailed basin delineation method because flows for the open/closed system analysis are needed on a per-inlet basis. This more detailed analysis is necessary, due to the sensitivity of smaller pipe networks, and for the results to be valid in short reaches of the stormwater system with quickly changing pipe capacities.

The difference in these two modeling approaches is that the "large scale hydrology" is conservative, as it places the peak flow that is expected at the bottom of the basin, at the top of the basin for modeling purposes, while the "small scale hydrology" places the flow on a per-inlet basis. The flow in the "small scale hydrology" is placed where it is expected to enter the system, and gives a detailed understanding of the actual conditions in the system at that particular flow entry location.

Automated routines in DWM were used to create a skeleton (spatially-correct base plan view) HEC-HMS model in the 50-acre scale for each of the four major watershed regions. The preliminary HMS model required basin area, curve numbers, lag times and reach lengths populated for each subbasin. A complete HMS model is created by adding 8-point routing cross section data for each routing reach, and assigning Manning's n values for channels with the aid of aerial photos and field data. Elevation-Area routing parameters are manually assigned for reaches with level pool routing as the main source of attenuation such as subbasin WC5 in Walnut Creek watershed model.

The meteorologic model used in HMS was the SCS Hypothetical type II storm for the 2-, 10-, 25-, 50-, and 100-year design storm events along with the 10-year/1-hour storm events with rainfall depths obtained from the Raleigh Stormwater Drainage Design Manual for Wake County. The control specifications for the simulation are one minute time steps and the duration of rainfall is 24-hours.

The simulation was run to obtain the 50-acre scale discharges (Appendix C - 8) for the various storm events required for the water surface profile analysis of the open system for each of the four major watersheds.

Computed discharges for 10-acre scale subbasins (Appendix C - 9) are required for hydraulic modeling of the stormwater inventory within the TCAP, and discharges for the inlet drainage areas were primarily

used to model the TCAP closed system inventory at priority locations designated by the Town of Cary. The 2- and 10-year storms along with the 10-year/1-hour storms were used for this analysis. The 10-year/1-hour storm was determined to be the critical storm for the systems that were tested, and is meant to represent a typical summer storm that causes the existing systems to approach critical capacity. This determination was based off of a short study done to determine which storm would cause the downstream portion of the system to run "full", and have this condition met for the majority of the systems studied.

Discharges for the 10-acre scale subbasins and inlet drainage areas were obtained by determining what portion of each of the 50-acre scale HEC-HMS models' discharges contributed to each 10-acre basin. The contributed flow from each 50-acre basin was calculated based on a combination of the percentage of impervious area and pervious area that each 10-acre basin comprised, compared to the total pervious area and impervious area in each 50-acre basin.

2.4 Impervious Surfaces

Impervious surfaces represent areas that allow little to no infiltration. The impervious surfaces layer was created from the Town of Cary's planimetric data and it includes most significant impervious surfaces such as roads, parking lots, building footprints, etc. The impervious surfaces layer was spatially intersected with the subbasins in GIS to estimate the total impervious area in each subbasin. The total impervious area per basin was used in the calculation of discharges for the "small scale hydrology" modeling.

2.5 Quality Control

Model comparison refers to comparing the results of a given study to previous/parallel studies and/or known results (e.g. gage data). This is often done to ensure that model results are reasonable and/or representative of known conditions. Differences in model results do not necessarily indicate that a given model is incorrect, especially for hydrologic modeling, where there are many variables (e.g. scale of study, model, design precipitation, storm event, etc.) and inherent assumptions.

Two sources of previous studies and alternative methods were used for the model comparison to create the flow comparisons found in Appendix C - 10, and are as follows:

1. Effective FEMA Analysis: Hydrologic and hydraulic modeling used to create effective FEMA maps for Wake County. Swift Creek watershed and Walnut Creek watershed were represented by two (2) subbasins in the NCFPM HEC-HMS models.
2. Urban Regression Equations: Blue Ridge-Piedmont Urban regression equations developed by USGS Report # 96-4084 (1996). The average percent impervious for each overall basin was used as input as shown in Appendix C - 10.

The comparisons generally show that flows determined for this study are greater than the flows found in the effective studies and those calculated using the urban regression equation. This is typical of the detailed methodology used to determine flows in this study and results in conservative conclusions for existing and future predicted flooding. Methods used in this study are generally within 46 percent of the results determined using urban regression equations, which is consistent with the standard predictive error of regression equations, and within 11 percent of results of independently derived effective FEMA/State studies.

In addition to the review of the overall results, additional quality control efforts were taken during development of each HEC-HMS input and development of the 10-acre hydrology. These efforts include,

but are not limited to, review and generation of alternative Tc flow path locations and conservative inflow location determinations when multiple inlets receive discharge from a basin.

2.6 Basin Hydrology Notes

The hydrologic modeling indicated a few features that are unique to the individual basins. The Coles Branch watershed develops a more significant percent of discharge in the headwater portions of the watershed than the other basins. Walnut Creek is not very sensitive to changes in time of concentrations of individual subbasins while the Swift Creek Tributary 7 basin shows immediate response to small changes to time of concentration in almost all subbasins. It was noted that development in Walnut Creek appeared to be the oldest development based on the development pattern of sub-divisions streets placed perpendicular to streams while the other basins, Coles Branch in particular, is developed streets and structures parallel to streams, which is nsistent with more modern setback and buffer rules.

3.0 Water Surface Profile Analysis and Floodplain Mapping

Hydraulic analysis on the study streams shown in Table 4 were completed to produce water surface profiles and 100-year floodplains, using techniques consistent with FEMA and NCFMP standards. The FEMA/NCFMP standard analysis was modified to exclude the 0.2% annual chance flood events and floodway analysis from the scope of work. The FEMA accepted open channel hydraulics model, HEC-RAS 3.1.2, as based on the FEMA August 17, 2004 Memorandum, was used to develop the open channel hydraulics. More than one version of HEC-RAS is accepted by FEMA currently, however, HEC-RAS 3.1.2 was selected to be consistent with on-going City of Raleigh studies, which include the downstream Walnut Creek basin. The results of the hydraulic modeling and profile development were enhanced by using automated floodplain mapping programs in GIS to visualize the spatial extents of the 1% (100-year) annual chance events flooding. The digital files were generated in MapInfo GIS and Auto CAD formats and submitted to the Town on January 10, 2006.

For the purposes of assessing flood hazards, hydraulic analysis involves the computation of flood elevations/depths and flow velocities. In simplistic terms, the hydraulic analyses transform the peak discharges computed by the hydrologic analyses to flood elevations, which are used to produce the resultant flood profiles and floodplains. For the study area, hydraulic analyses were performed along the streams shown in Table 4 to determine the existing flood profiles for the flooding events having a 50-, 10-, 4-, 2-, and 1- percent annual chance of being equaled or exceeded in any given year.

Table 4 Study Streams

Stream Name	Length (miles)
Coles Branch	1.51
Swift Creek Tributary 7	1.59
Walnut Creek	0.98
Walnut Creek Tributary	0.32

3.1 Modeling Approach

FEMA and NCFMP standards were used as a template for creating the hydraulic models, which were used to develop the water surface profiles and 100-year floodplains. The hydraulic analyses associated with the flood hazard assessment focused on the 100-year recurrence interval, also known as the base flood, which FEMA uses as a national standard for floodplain management and flood insurance rating purposes. HEC-RAS 3.1.2 was the hydraulic model used, since it is an approved model for flood studies on open channels according to FEMA and NCFPM. Inputs in the hydraulic models that use FEMA guidance include boundary conditions, Manning's n values, expansion/contraction coefficients, ineffective flow areas, etc. Hydraulic parameters such as Manning's roughness coefficients and definition of ineffective flow areas were generally based on a 100-year flood event.

Debris blockage of structures can have a significant impact on upstream flooding, but is typically not included in flood hazard assessments due to the lack of historical documentation. The hydraulic analyses contained in this study are based on unobstructed flow.

3.2 Hydraulic Data and Model Development

The HEC-RAS hydraulic model was used to analyze four different flooding sources covering approximately 4.4 stream miles in the study area. Table 5 lists the flooding sources and the upstream and

downstream limits of study. A visual representation of the areas studied can be viewed in Appendix D - 4.

Table 5 Limits of Detail Study

Flooding Source	Limits of Study
Coles Branch	From just east of Galveston Ct. and West St. to south of Maynard Rd.
Swift Creek Tributary 7	From west of Page St. and north of Harrison Ave. to just north of Brookgreen Dr.
Walnut Creek	From just north of Meadow Dr. to 200 feet east of Clay St.
Walnut Creek Tributary	From north-west of Ralph Dr. to its confluence with Walnut Creek

HEC-RAS requires a number of parameters and data inputs to compute flood profiles, including:

1. Peak discharges
2. Downstream starting conditions for sub-critical flow analyses
3. Stream valley cross section data
4. Roughness coefficients for friction losses
5. Bridge and culvert crossing data - geometry and dimensions of the openings, top-of-road profile, and entrance characteristics
6. Other hydraulic characteristics such as expansion and contraction loss coefficients and ineffective flow areas

3.3 Peak Discharges

The peak discharges used in the HEC-RAS model were obtained from the results of the detailed hydrologic analyses and are provided in Appendix D - 1.

3.4 Reach Boundary Conditions

The starting water surface elevations (WSEs) for the 10-, 50-, 100-, and 500-year recurrence interval storms for Coles Branch and Walnut Creek were taken from the Effective Flood Insurance Study dated April 6th of 2005. The 2- and 25-year starting WSEs were determined by engineering judgment. Only the starting WSEs for the 10-, 100-, and 500-year recurrence interval storms were available for Swift Creek Tributary 7, so engineering judgment was used to determine the 2-, 25-, and 50-year starting WSEs. Walnut Creek Tributary confluences with Walnut Creek, so it was modeled using a normal depth water surface slope of 0.0096. The WSE of Walnut Creek Tributary at its confluence is less than that of Walnut Creek, thus Walnut Creek Tributary is under backwater for a distance on the downstream portion of the reach. Boundary conditions are listed in Appendix D - 2.

3.5 Cross Sections

Stream valley cross section data for the HEC-RAS model are from a combination of the following sources:

- Digital terrain data created using the Town's 2 foot contour data
- Field surveys conducted by Dewberry

A total of 152 cross sections were used to represent the stream valley terrain and flow obstructions such as bridges and culverts in the hydraulic analysis. The majority of the cross section data was obtained from the Town's Pilot and Maynard Loop Stormwater Inventory Database, which includes a combination of field-surveyed points and elevation data extracted from the digital terrain model. In the vicinity of bridge and culvert crossings, top-of-road profiles and channel geometry was obtained from field surveys conducted by Dewberry. Channel cross-sections using relative vertical elevations along Walnut Creek, Coles Branch and Swift Creek were also collected. The relative vertical elevation cross-sections were collected at intervals of 1500 to 2000 feet along the three streams modeled. This data was combined with the Town's existing topographic information to generate representative cross-section data for the HEC-RAS modeling.

HEC-RAS cross sections were located perpendicular to flow on average every 183 feet along the channel as well as upstream and downstream of road crossings and confluences, and at major changes in stream valley characteristics such as channel slope, roughness, or geometry. Once the locations of the cross sections were established, automated routines were used to extract ground profiles (station-elevation data) along the cross sections and write the data to a HEC-RAS compatible format by utilizing an extension in ArcMap called Geo-RAS.

3.6 Manning's n Values

The hydraulic roughness coefficients (Manning's n-values) are a measure of the stream valley's resistance to flow, or in other words, the friction losses. Stream valleys with high roughness coefficients, for example channels with large boulders and densely forested overbanks, will tend to slow the flow and result in higher flood elevations, while relatively smooth areas, like mowed grass provide little resistance to flow and result in higher velocities and lower flood elevations. Typical factors that influence the resistance to flow, or friction losses include:

- Degree of channel meander and irregularity
- Type and density of vegetation along the channel and floodplain
- The size and shape of the channel and floodplain
- Number of obstructions in the channel and floodplain
- Flood elevation and discharge
- Depth of flow

Table 6 summarizes the range of Manning's n-values used for each stream. Manning's n-values were estimated based on best engineering judgment and field observations of the channels and floodplain areas.

Table 6 Manning's n Ranges

Flooding Source	Channel	Overbanks
Coles Branch	0.045	0.016-0.15
Swift Creek Tributary 7	0.045	0.045-0.1
Walnut Creek	0.04-0.045	0.07-0.15
Walnut Creek Tributary	0.045	0.08

3.7 Stream Crossings

In urban areas, bridge and culvert crossings can often be the cause of flooding due to inadequate capacity. Therefore, it is important to properly model bridges and culverts so that the effects of these structures on flood elevations can be accurately determined. The bridge data required for HEC-RAS modeling includes:

- Opening geometry including abutment side slopes, low-chord elevations, pier shape and size, number of spans, and width of bridge decking
- Top-of-road profile along the highest point on the road
- Type of railing and its geometry
- Contraction and expansion characteristics

The culvert data required for HEC-RAS modeling includes:

- Inlet and outlet conditions
- Culvert type, material, shape, length, number of barrels, and dimensions
- Upstream and downstream invert elevations
- Top-of-road profile along the highest point on the road
- Type of railing and its geometry
- Contraction and expansion characteristics

This data was previously collected as part of the Town's Pilot and Maynard Loop Stormwater Inventory Database and was supplemented with additional survey as needed. Additionally, detail survey was collected at all crossings, which included top of road, opening information and upstream downstream channel cross-sections. All crossings modeled in the HEC-RAS analysis can be found in Table 7.

Table 7 Crossings modeled in HEC-RAS

Coles Branch			
Crossing	**Number**	**Size (ft)**	**Material**
Railroad	1	2.5 x 3	RCP
Dirt Parking Lot	1	4	RCP
Dixon	1	5 x 3.25	CMP
Access Road	1	7.5 x 6	CMP
Oscar	3	5	CMP
Madison	2	7	CMP
Carrousel	3	6.5	CMP

Swift Creek			
Crossing	**Number**	**Size (ft)**	**Material**
West	1	2.5	RCP
Heater	2	1	RCP
Dixon	1	3	RCP
Glendale	3	3.75	RCP
Glendale	3	3.75	RCP
Wooden Footbridge	1	37 ft wide	Wood
Maynard	2	6	RCP

Walnut Creek			
Crossing	**Number**	**Size (ft)**	**Material**
Dowell	1	3	RCP
Byrum	2	3.5	RCP
Walnut	2	3.5	RCP
Fairview	2	3.5	RCP
E. Park	2	3.5	RCP
Footbridge	1	24 ft wide	Wood
Webster	1	12 x 4	RCP
Urban	1	12 x 4	RCP
Clay	2	9 x 5	RCP
Warren	1	4	RCP

3.8 Ineffective Flow Areas

Areas of the floodplain that do not convey flow are modeled with ineffective flow areas. Ineffective flow areas were designated in the HEC-RAS model where natural areas of high ground or ridges along the

overbanks tend to constrict flow or prevent flow on the landward side of the ridge. Ineffective flow areas were also defined upstream and downstream of bridge and culvert crossings based generally on a 1:1 contraction ratio upstream and a 2:1 expansion ratio downstream as recommended in the HEC-RAS, Hydraulic Reference Manual (USACE, 2002).

3.9 Expansion and Contraction Coefficients

Expansion and contraction loss coefficients are used to approximate the energy loss between cross sections and through bridge and culvert crossings. Typically, the greatest expansion loss occurs downstream of a bridge or culvert, as flow expands from the relatively narrow width of the bridge or culvert to the full width of the floodplain. Similarly, the greatest contraction loss occurs upstream of a bridge or culvert, as flow contracts from the full width of the floodplain to the narrower bridge or culvert opening. Expansion and contraction loss coefficients were defined to be 0.3 and 0.5, respectively. These values were chosen in an effort to be consistent with on-going NCFMP studies and are generally in accordance with procedures outlined in HEC-RAS, Hydraulic Reference Manual, (USACE, 2002).

3.10 Summary of Hydraulic Results

To assess existing flooding conditions, hydraulic analyses were conducted to predict flood elevations (Appendix D - 3) for floods having a recurrence interval of 2-, 10-, 25-, 50-, and 100-years. The number of structures in each basin that may experience flooding during the 100-year storm event is provided in Table 8, and the level of service for roadways in the studied area is shown in Table 9. The Town of Cary design standards maintain a 25-year level of service for roadways and do not allow structures to be built in the 100-year floodplain.

It was determined that a total of 146 structures, with a total tax value of nearly $33,000,000 may experience flooding in the 100-year storm event. The flood risk was determined based on spatial location of the structures only, as elevation data was not available for this analysis. It is anticipated the future development in the basins may increase the number and/or frequency of structures that experience flood risks. Additionally, there are 75 undeveloped parcels that are at least partially impacted by the 100-year floodplain.

Model results indicate that 88 percent of the crossings along the streams modeled do not meet the Town of Cary's desired level of service and current design standard. Five (5) of the crossings of the 24 studied are a railroad, parking lot, access road, and two footbridges. When the small crossings are not considered, 89 percent of the roads along the modeled streams do not meet the desired level of service.

A color map of the floodplain boundary along with a list of floodprone structures and undeveloped or partially developed floodprone parcels along each studied stream can be found in Appendix D - 4. This data can be useful to the Town in fostering informed development decisions. Undeveloped floodprone parcels can be highlighted by the Town, and critically evaluated prior to development to reduce the chance of increasing upstream or downstream flood risks.

Table 8 Number of Floodprone Structures Per Basin

	Number of Floodprone Structures
Coles Branch	31
Swift Creek	48
Walnut Creek	67

Table 9 Level of Service Table

Coles Branch		Swift Creek		Walnut Creek	
Crossing	**Overtopping Profile (YR)**	**Crossing**	**Overtopping Profile (YR)**	**Crossing**	**Overtopping Profile (YR)**
Railroad	10	West	2	Dowell	2
Dirt Parking Lot	2	Heater	2	Byrum	10
Dixon	2	Dixon	2	Walnut	10
Access Road	100	Glendale	2	Fairview	2
Oscar	25	Glendale	2	Park	10
Madison	50	Wooden Footbridge	2	Footbridge	2
Carrousel	NONE	Maynard	10	Webster	10
				Urban	10
				Clay	10
				Warren	25

3.11 Quality Control of Hydraulic Modeling and Mapping

Review of hydraulic models was completed to confirm that modeling approaches and data development were done in conformance with NCFMP "detailed" review methods. This process included a submission of the model by a modeler to a reviewer, who reviewed the models for consistency with FEMA and NCFMP standards. For this study the review was performed by Kenneth W. Ashe, P.E., CFM the project manager for this study as well as lead QA/QC Reviewer for the NCFMP. Comments and questions were re-submitted to the modeler for corrections, and this process was done until the reviewer had no outstanding issues with the created model. Reviews of models that do not include a floodway profile require checks on the following (This is not an extensive list, but includes many variables considered):

- Boundary conditions
- Ineffective flow areas
- Manning's n values and blocked obstructions used
- Entrance/exit loss coefficients
- Culverts and bridges
- Expansion/contraction coefficients
- Channel bank placement
- Survey and Tin data in cross sections
- Discharge check (flow change locations and quantity)
- Velocities
- Energy grade line fluxes
- Crossing profiles and profile irregularities
- Floodplain breaks or missed structures
- Proper cross section placement

3.12 Discussion of Hydraulic Results

Section 3.10 indicates a significant amount of existing and potential structure and property is located within the 100-year floodplains of the three (3) reaches studied as part of this project. With the development of the 100-year water surface elevation for each of the stream, the Town now has a tool to guide development beyond customary setbacks and buffers for the undeveloped lots in these areas. It is recommended that the Town require new development and substantial improvement be elevated above the 100-year floodplain, consistent with the Town's existing flood damage prevention ordinance, to avoid placing what may be several tens of millions of dollars of property at risk for flooding.

Additionally, the Town should compare finished floor elevations (FFE) of existing structures in this area to the 100-year water surface elevations to evaluate the overall potential flood risk. It is recommended that the Town acquire elevation certificates for any of these structures lacking FFE data and that substantial improvement be required to adhere to the Town's existing flood damage prevention ordinance.

3.13 Roadway Level of Service Analysis Summary of Results

The roadway level of service results, listed in Table 8, from the Water Surface Profile Analysis indicate that several roadways do not meet the Level of Service Criteria set out by the Town of Cary. The Town of Cary's design manual calls for all roadways to provide a 25-year storm event level of service (100-year for those roads within the 100-year floodplain). This was extended to apply to the newly developed floodplain information from this study.. A 25-year storm event level of service alternative for all roadways was also investigated.

A cursory investigation was performed for each crossing to determine the depth of flooding and the relationship to the stream system as a whole. When a roadway was overtopping and acting as a weir the amount of weir flow was assessed. An initial estimate was obtained by using the orifice equation to account for the loss of head due to removing weir flow and ultimately to provide an approximate additional area to convey flow under the roadway while maintaining a 100-year roadway level of service. After the initial estimate of additional area was completed, HEC-RAS model runs were performed in order to evaluate the validity and feasibility of the culvert sizing Many culverts were flooded due to backwater conditions. In cases where downstream improvements resolved the backwater this was noted. In cases where backwater persisted and evaluation of the potential partial improvement of the culvert was pursued. In some cases this resulted in 25-year LOS improvements only, in others while a marked improvement could be achieved, neither the 25 nor 100-year LOS were attainable. Finally, several crossings were unable to achieve the desired level of service due to the topology of the area and existing development.

3.13.1 Swift Creek Tributary 7

Swift Creek Tributary 7 has six roadway crossings that were investigated for the purpose of this analysis: West, Heater, Dixon, Glendale[3], Glendale[4], and Maynard. Of these, Maynard was the most functional, operating at a 2 year level of service. Table 10 summarizes the proposed flow area upgrades and their result to the level of service as it relates to the 100-year storm event. A series of more detailed summary tables for both the 100-year storm event and the 25-year storm event can be found on page 19. Table 11

on page 19 summarizes proposed flow areas, proposed pipe sizes, and cost estimations for individual road crossings.

Table 10 Level of Service Table for Swift Creek Tributary 7 (100 yr)

Crossing	Flow Area of Existing Culvert(sq. ft)	Flow Area of Proposed Culvert(sq. ft)	Overtopping (ft/100yr LOS)
West[1]	5	***120***	0.01
Heater	8	114	100yr LOS
Dixon[2]	7	***NA***	***NA***
Glendale[3]	53	***228***	0.5
Glendale[4]	53	***228***	0.48
Maynard	57	260	100yr LOS

After initial additional culvert area estimates, the HEC-RAS model was used to determine the actual roadway level of service. A 100-year level of service was achieved for Maynard and Heater. West still overtops by 0.01-ft, but would provide a 25-year level of service. During analysis it was determined that any increase in the pipe diameter at Dixon Ave. would have little effect on the roadway level of service due to the surrounding topology. As a result, no upgrades were recommended for this crossing and no improved level of service was achieved. Glendale is a horse shoe shaped road that crosses Swift Creek Tributary 7 twice. While a resolution to the road overtopping does not appear feasible a significant reduction in flooding depth can be achieved. During the 100-year event the water surface profile is reduced by 2.5 feet, but still exceeds the top of roadway by 0.5-ft for Glendale[3] (the most upstream of the two Glendale's) and by 2 feet resulting in 0.48-ft of flood depth for Glendale[4]. The 25-year analysis indicated that both Glendale crossing would overtop during the 25-year event. .

Table 11 Detailed Level of Service Results for Swift Creek Watershed

25 Year Roadway Level of Service Results

Crossing	Number of barrels	Type	Existing Size (ft)	Existing Material	Existing Culvert Length (ft)	Flow Area of Existing Culvert(sq. ft)	Additional flow area used in the model (sq.ft)	Total flow area used in the model.(sq.ft)	Alternate Size(ftXft)	Cost Estimation				
										Quantity	Rate	Amount	Add 100%(contingency)	Final Amount
West	1	Culvert	2.5	RCP	190	5	115	120	(5)4x6*	5	$ 752	$ 714,400	$ 714,400	$ 1,428,800
Heater	2	Culvert	1 and 3	RCP	57	8	87	95	(4) 4X6	4	$ 752	$ 171,456	$ 171,456	$ 342,912
Dixon[2]	1	Culvert	3	RCP	125	7	***NA***	***NA***						
Glendale[3]	3	Culvert	6 X 3.75	RCP	50	53	***175***	***228***	***(4) 10x6****	4	$ 1,612	$ 322,400	$ 322,400	$ 644,800
Glendale[4]	3	Culvert	6 X 3.75	RCP	49	53	***175***	***228***	***(4) 10x6***	4	$ 1,612	$ 315,952	$ 315,952	$ 631,904
Maynard	2	Culvert	6	RCP	79	57	143	200	(4) 10 X5	4	$ 1,612	$ 509,392	$ 509,392	$ 1,018,784
														$ 4,067,200

100 Year Roadway Level of Service Results

Crossing	Number of barrels	Type	Existing Size (ft)	Existing Material	Existing Culvert Length (ft)	Flow Area of Existing Culvert(sq. ft)	Additional flow area used in the model (sq.ft)	Total flow area used in the model.(sq.ft)	Alternate Size	Cost Estimation				
										Quantity	Rate	Amount	Add 100%(contingency)	Final Amount
West[1]	1	Culvert	2.5	RCP	190	5	***115***	***120***	***(5) 4x6****	5	$ 752	$ 714,400	$ 714,400	$ 1,428,800
Heater	2	Culvert	1 and 3	RCP	57	8	106	114	(4) 4X7	4	$ 1,202	$ 274,056	$ 274,056	$ 548,112
Dixon[2]	1	Culvert	3	RCP	125	7	***NA***	***NA***						
Glendale[3]	3	Culvert	6 X 3.75	RCP	50	53	***175***	***228***	***(4) 10 X6****	4	$ 1,612	$ 322,400	$ 322,400	$ 644,800
Glendale[4]	3	Culvert	6 X 3.75	RCP	49	53	***175***	***228***	***(4) 10X 6***	4	$ 1,612	$ 315,952	$ 315,952	$ 631,904
Maynard	2	Culvert	6	RCP	79	57	203	260	(4) 12X6	4	$ 1,886	$ 595,976	$ 595,976	$ 1,191,952
														$ 4,445,568

Notes:

- * Indicates channel widening/improvements
- 1 Solutions area gives a ponding depth of 0.01ft
- 2 No improvements in the culvert because the crown is above the fill.
- 3 Solution area gives a ponding depth of 0.32ft for 25 years and ponding depth of 0.5ft for 100 year event and it will require channel improvements.(Initial ponding 2.59ft and 3.15ft)
- 4 Solution area gives a ponding depth of 0.21ft for 25 years and ponding depth of 0.48ft for 100 year event and it will require channel improvements.(Initial ponding 2.13 and 2.55ft)

3.13.2 Coles Branch

Coles Branch has seven roadway/parking lot crossings that were investigated for the purpose of this analysis: the Railroad, a Dirt Parking Lot, Dixon, an Access Road, Oscar, Madison, and Carrousel. Of these, Carrousel was the most functional, operating at a 100-year level of service. Table 12 summarizes the proposed flow area upgrades and their result to the level of service as it relates to the 100-year storm event. A series of more detailed summary tables for both the 100-year storm event and the 25-year storm event can be found on page 21. Table 14 on page 21 summarizes proposed flow areas, proposed pipe sizes, and cost estimations for individual road crossings.

Table 12 Level of Service Table for Coles Branch (100 yr)

Crossing	Flow Area of Existing Culvert(sq. ft)	Flow Area of Proposed Culvert(sq. ft)	Overtopping (ft/100yr LOS)
Railroad	8	23	N/A
Dirt Parking Lot	13	NA	N/A
Dixon[b]	13	***72***	0.01
Access Road	35	48	100yr LOS
Oscar	59	105	100yr LOS
Madison	77	112	100yr LOS
Carrousel	99	NA	100yr LOS

A 100-year level of service was achieved for the Access Road, Oscar, and Madison. As previously stated, Carrousel already had a 100-year level of service. Due to discussions with the Town and the high costs in upgrading railroad crossings no alternatives were investigated for the railroad crossing in the upstream portion of the studied stream. The dirt parking lot at 318 Dixon Ave., did not have the required topology to facility level of service improvements. This is a private drive and without extensive earth work little can be done to reduce flood risk. Dixon Ave. has less than a 100-year level of service due to the topology of the area. However, a significant reduction in flooding depth can be achieved. During the 100-year event the water surface profile is reduced by 1.99-ft resulting in a flooding depth of only 0.01-ft. Dixon roadway will have a 25-year level of service as currently modeled.

Table 13 Detailed Level of Service Results for Coles Creek Watershed

25 Year Roadway Level of Service Results

Crossing	Number of barrels	Type	Existing Size (ft)	Existing Material	Existing Culvert Length (ft)	Flow Area of Existing Culvert (sq. ft)	Additional flow area used in the model (sq.ft)	Total flow area used in the model.(sq.ft)	Appropriate sizes same as existing culvert	Cost Estimation				
										Quantity	Rate	Amount	Add 100%(contingency)	Final Amount
Railroad	1	Culvert	2.5 x 3(Box)	RCP	61	8	7	14	2.5X3;					
Dirt Parking Lot[1]	1	Culvert	4	RCP	84	13	NA	NA						
Dixon[a]	1	Culvert	5 x 3.25(Ellipse)	CMP	40	13	***47***	***60***	***(4) 5X3.25*;***	4	$ 240	$ 38,246	$ 38,246	$ 76,493
Access Road	1	Culvert	7.5 x 6(Ellipse)	CMP	39	35	NA	NA						
Oscar	3	Culvert	5	CMP	62	59	4	63	30inch	1	$ 151	$ 9,434	$ 9,434	$ 18,869
Madison	2	Culvert	7	CMP	97	77	NA	NA						
Carrousel	3	Culvert	6.5	CMP	90	99	NA	NA						
													Total Amount	**$ 95,362**

100 Year Roadway Level of Service Results

Crossing	Number of barrels	Type	Existing Size (ft)	Existing Material	Existing Culvert Length (ft)	Flow Area of Existing Culvert (sq. ft)	Additional flow area used in the model (sq.ft)	Total flow area used in the model.(sq.ft)	Appropriate sizes same as existing culvert	Cost Estimation				
										Quantity	Rate	Amount	Add 100%(contingency)	Final Amount
Railroad	1	Culvert	2.5 x 3(Box)	RCP	61	8	15	23	(2)2.5X3;					
Dirt Parking Lot	1	Culvert	4	RCP	84	13	NA	NA						
Dixon[b]	1	Culvert	5 x 3.25(Ellipse)	CMP	40	13	***59***	***72***	***4 (5X3.25)*;***	4	$ 240	$ 38,246	$ 38,246	$ 76,493
Access Road	1	Culvert	7.5 x 6(Ellipse)	CMP	39	35	13	48	5 x 3.25(Ellipse)	1	$ 240	$ 9,346	$ 9,346	$ 18,691
Oscar	3	Culvert	5	CMP	62	59	46	105	(3) 5 *	3	$ 328	$ 61,480	$ 61,480	$ 122,961
Madison	2	Culvert	7	CMP	97	77	35	112	7	1	$ 781	$ 75,991	$ 75,991	$ 151,983
Carrousel	3	Culvert	6.5	CMP	90	99	NA	NA						
													Total Amount	**$ 218,145**

Notes:

- * Indicates channel widening/improvements
- 1 No improvements in the culvert because the crown is above the fill.
- a Solutions area gives a ponding depth of 0.01ft
- b Solutions area gives a ponding depth of 0.01ft and 0 ponding depth for 25 year event and it will require channel improvements.

3.13.3 Walnut Creek

Walnut Creek has nine roadway crossings that were investigated for the purpose of this analysis: Dowell, Byrum, Walnut, Fairview, E. Park, Webster, Urban, Clay, and Warren. Of these, Warren was the most functional, operating at a 10-year level of service. Table 14 summarizes the proposed flow area upgrades and their result to the level of service as it relates to the 100-year storm event. . A series of more detailed summary tables for both the 100 year storm event and the 25-year storm event can be found on page 23. Table 15 on page 23 summarizes proposed flow areas, proposed pipe sizes, and cost estimations for individual road crossings.

Table 14 Level of Service Table for Walnut Creek (100 yr)

Crossing	Flow Area of Existing Culvert(sq. ft)	Flow Area of Proposed Culvert(sq. ft)	Overtopping (ft/100yr LOS)
Dowell[1]	7	***53***	0.08
Byrum[2]	19	***80***	0.01
Walnut	19	105	100yr LOS
Fairview	19	152	100yr LOS
E. Park[3]	19	***NA***	NA
Webster[4]	48	***NA***	>1.00
Urban[5]	48	***NA***	>2.00
Clay[6]	90	***187***	0.01
Warren	13	24	100yr LOS

A 100-year level of service was achieved for Walnut, Fairview, and Warren. Several crossings, E. Park, Webster, and Urban, require significant channel improvements and earth work in order to provide an alternative that allows a 100-year level of service. As a result, E. Park was not upgraded in the model because no significant increase in level of service could be achieved. Webster was upgraded so that during the 100-year and 25-year events the roadway overtops by more than 1.00-ft and by 0.21-ft respectively. Urban was upgraded, but during the 100-year and 25-year events the roadway still overtops by more than 2.00-ft and by 0.52-ft respectively. Other crossings that significantly reduce flood risk, but do not achieve a 100-year level of service include Dowell, Byrum, and Clay Dowell, Byrum, and Clay had 100-year flood reductions of approximately 1.0-ft, 0.9-ft, and 1.2-ft respectively resulting in roadway flooding depths of 0.08-ft, 0.01-ft, and 0.01-ft respectively for the 100-year storm event. Dowell, Byrum, and Clay will all maintain a 25-year level of service as modeled.

Table 15 Detailed Level of Service Results for Walnut Creek Watershed

25 Year Roadway Level of Service Results

Crossing	Number of barrels	Type	Existing Size (ft)	Existing Material	Existing Culvert Length (ft)	Flow Area of Existing Culvert(sq. ft)	Additional flow area used in the model (sq.ft)	Total flow area used in the model.(sq.ft)	Appropriate sizes same as existing culvert	Cost Estimation				
										Quantity	Rate	Amount	Add 100%(contingency)	Final Amount
Dowell	1	Culvert	3	RCP	99	7	29	36	(4) 3 X 3*	4	$ 684	$ 271,384	$ 271,384	$ 542,768
Byrum	2	Culvert	3.5	RCP	50	19	33	52.5	(2) 4X7*	2	$ 1,202	$ 120,753	$ 120,753	$ 241,506
Walnut	2	Culvert	3.5	RCP	72	19	33	52.5	(2) 4X7*	2	$ 1,202	$ 173,401	$ 173,401	$ 346,801
Fairview	2	Culvert	3.5	RCP	77	19	61	80	(4) 4X5	4	$ 752	$ 230,744	$ 230,744	$ 461,487
E. Park[3]	2	Culvert	3.5	RCP	53	19	***NA***	***NA***						
Webster[4]	1	Culvert	12 x 4	RCP	61	48	***177***	***225***	***(4) 12 X 4****	4	$ 1,706	$ 418,106	$ 418,106	$ 836,213
Urban[5]	1	Culvert	12 x 4	RCP	46	48	***192***	***240***	***(4) 12 X 4****	4	$ 1,706	$ 314,996	$ 314,996	$ 629,992
Clay	2	Culvert	9 x 5	RCP	45	90	66	156	(2) 5X9*	2	$ 1,748	$ 157,425	$ 157,425	$ 314,850
Warren	1	Culvert	4	RCP	61	13	4	17	30 inch	1	$ 126	$ 7,667	$ 7,667	$ 15,334
													Total Amount	**$ 3,388,951**

100 Year Roadway Level of Service Results

Crossing	Number of barrels	Type	Existing Size (ft)	Existing Material	Existing Culvert Length (ft)	Flow Area of Existing Culvert(sq. ft)	Additional flow area used in the model (sq.ft)	Total flow area used in the model.(sq.ft)	Appropriate sizes same as existing culvert	Cost Estimation				
										Quantity	Rate	Amount	Add 100%(contingency)	Final Amount
Dowell[1]	1	Culvert	3	RCP	99	7	***45***	***53***	(6) 3 X 3*	6	$ 684	$ 407,076	$ 238,453	$ 645,529
Byrum[2]	2	Culvert	3.5	RCP	50	19	***61***	***80***	(2) 4 X10*	2	$ 1,462	$ 146,873	$ 146,873	$ 293,745
Walnut	2	Culvert	3.5	RCP	72	19	86	105	(5) 4X 5 *	5	$ 752	$ 271,209	$ 271,209	$ 542,418
Fairview	2	Culvert	3.5	RCP	77	19	133	152	(4) 4 X 10 *	4	$ 1,462	$ 448,600	$ 448,600	$ 897,200
E. Park[3]	2	Culvert	3.5	RCP	53	19	***NA***	***NA***						
Webster[4]	1	Culvert	12 x 4	RCP	61	48	***NA***	***NA***	***(4) 12 X 4****	4	$ 1,706	$ 418,106	$ 418,106	$ 836,213
Urban[5]	1	Culvert	12 x 4	RCP	46	48	***NA***	***NA***	***(4) 12 X 4****	4	$ 1,706	$ 314,996	$ 314,996	$ 629,992
Clay[6]	2	Culvert	9 x 5	RCP	45	90	***97***	***187***	(2) 5X9*	2	$ 1,748	$ 157,425	$ 157,425	$ 314,850
Warren	1	Culvert	4	RCP	61	13	11	24	4	1	$ 240	$ 14,604	$ 7,667	$ 22,271
													Total Amount	**$ 4,182,217**

Notes:

*	Indicates channel widening/improvements
Dowell[1]	Solutions area gives a ponding depth of 0.08ft for 100year event.
Byrum[2]	Solutions area gives a ponding depth of 0.01ft for 100 year event.
E. Park[3]	Changing only culvert size is not sufficient for improvements.
Webster[4]	Solution area gives a ponding depth of 0.21ft for 25 years and ponding depth over a foot for 100 year event and it will require channel improvements.(Initial ponding 0.97ft and 1.42ft)
Urban[5]	Solution area gives a ponding depth of 0.52ft for 25 years and ponding depth over 2 ft for 100 year event and it will require channel improvements.(Initial ponding 1.68ft and 2.24ft).
Clay[6]	Solutions area gives a ponding depth of 0.01ft for 100 year event.

3.14 Roadway Level of Service Analysis Discussion of Results

Section 3.13 indicates that several roadway crossings that do not currently provide a 100-year level of service could be upgraded to meet that demand. The few times a 100-year level of service is not feasible for a crossing a 25-year level of service can usually be achieved. Only 7 crossings analyzed could not achieve a 25-year level of service with culvert size upgrades and minor channel modifications. The seven crossings include E. Park, Webster, and Urban from the Walnut Creek watershed, Dixon from the Coles Branch watershed, and Dixon and the Dirt Parking lot in the Swift Creek watershed. It may be possible to perform more significant upgrades in order to achieve the desired level of service, but they were not investigated for the purposes of this report. A summary of the overall cost for each watershed is provided in Table 12. The table provides summary estimates to provide a 25-year level of service and a 100-year level of service to individual watersheds. An individual crossing summary spreadsheet for cost and several other variables was presented in section 3.13 with the individual stream reaches.

Table 16 Cost Estimate Summary for Level of Service Improvements

Area Description	Event	
	25-Year LOS	100-Year LOS
Coles Branch	$ 95,362	$ 218,145
Swift	$ 4,067,200	$ 4,445,568
Walnut	$ 3,388,951	$ 4,182,217
Total Cost	$ 7,551,512	$ 8,845,929

4.0 Hydraulic Grade Line Analysis

Hydraulic grade line analyses were completed on existing stormwater systems with target 10 acre drainage areas that serve as tributaries to Walnut Creek, Coles Branch and Swift Creek Tributary 7. The trunk lines modeled were chosen based on dialogue with the Town to assure that the main skeleton (primary conveyance) of the stormwater system is properly sized. Existing and ultimate build-out conditions within the TCAP area were modeled as separate scenarios for each drainage system. The hydraulic grade line analysis highlights areas that are at flood risk in the existing and future condition models. The results of this study will allow the Town to prioritize the upgrade of systems that are currently inadequate and those that will likely become inadequate in the future based on anticipated land use changes in the TCAP area.

For the purposes of assessing flood hazards in areas that require detailed open and closed system analysis in unison, Bentley's Civil Storm was utilized. Civil Storm is a stormwater modeling program that allows interdependent stormwater systems to be modeled simultaneously. Concurrent modeling of stormwater systems allows for analysis of areas that may have backwater effects on one another. Within the TCAP area, hydraulic analyses were performed for the systems listed in Table 17 and shown in the HGL Index Map in Appendix A to determine both existing and ultimate development hydraulic grade line profiles for the 2- and 10-year 24-hour flood events along with the 10-year/1-hour flood event. The systems study limits are discussed in detail in section 4.8 Summary of Hydraulic Results this report.

Table 17 Hydraulic Grade Line Analysis Study Areas

Reach Name	Approximate Open System Length (Feet)	Approximate Closed System Length (Feet)
Area 1	393	1,286
Area 2	784	366
Area 3	682	1,302
Area 4	777	2,996
Area 5	848	315
Area 6	1,060	381
Area 7	63	586
Area 8	1,193	1,553
Area 9	2,691	1,758
Area 10	1,724	772
Area 11	787	232
Area 12	1,368	863
Area 13	515	2,507
Area 14	1,606	317
Area 15	43	1,626

4.1 Modeling Approach

Six storm durations/frequencies were chosen to be analyzed during the hydraulic grade line analysis; the existing and future 2- and 10-year storm events and the 10-year 1hour storm events. The critical storm is considered to be the 10-year/1-hour storm event, as it was determined to be the storm that represents a high intensity summer storm in the Town of Cary, which is expected to run the pipe at the most downstream end of a particular system at the pipes capacity. Pipe capacity is the maximum amount of flow a pipe can pass under non-pressure conditions (i.e. flow at pipe crown). The Town's desired level of service for the TCAP conveyance system is the 10-year 24-hour storm event. It was thus important to

model the 2- and 10-year storm events to show the frequency with which flooding may occur, and also show the degree that flooding may impact nearby structures and/or roads.

Debris blockage of structures can have a significant impact on upstream flooding, but is typically not included in flood hazard assessments due to the lack of historical documentation. The hydraulic analyses contained in this study are based on unobstructed flow. As a result, the HGL elevations computed in the Civil Storm models, given in tables found in Appendix F, and shown on the HGL profiles in Appendix G are considered valid only if the hydraulic structures remain unobstructed by debris. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.2 Hydraulic Data and Model Development

Stormwater inventory collected during the on-going Town of Cary Inventory Project has been utilized as the basis for producing hydraulic models that represent existing stormwater conditions in the TCAP area. The previously collected Maynard Loop and Pilot inventory data provides detailed information for stormwater structures, but only limited detail regarding the dimensions of the open flow channels throughout the project. Therefore, 62 additional approximate channel cross sections were field collected using a GPS unit, a measuring rod, and a camera. Inventory and cross sectional data was analyzed and standardized in ArcMap to provide a basis for creating exportable data for use in Civil Storm.

Civil Storm allows for direct integration of the inventory data that has been standardized in ArcMap. To utilize this tool in Civil Storm; parameters such as sizes, shapes, elevations, node names, etc. were specified in ArcMap in a form that Civil Storm would recognize, which allowed for Civil Storm to directly reference these data sources to create its own database containing this information. Spatial referencing of this data, as provided in the inventory collection process, made the data spatially correct, which allowed Civil Storm to recognize start/stop nodes and lengths of pipes, culverts, or channels. Standardizing the data before entering Civil Storm so that it can be recognized and used by the model greatly reduced the chance of data input errors that may have occurred from manual input.

Values such as Manning's n, material, entrance/exit loss coefficients, and junction losses had to be specified manually in Civil Storm. During this manual process data was validated to verify that the imported data was correctly referenced. Cross sections were manually given station/elevation points if the channel collected in the field did not have enough detail required to provide dimensioning of the channel to potential flood elevations. Manholes and inlets that could not be accessed or verified by repeat field visits were given assumed dimensions based on the observed average of that particular structure type within the Town's inventory.

Existing and future flow models were set up using hydrology that was produced for each individual inlet or cross section in the system. This allowed for a detailed analysis of how each inlet or channel would operate during different flow conditions. To maintain flow continuity in the modeled systems, overflow at inlets is captured by a theoretical gutter system that has been created to mimic real world conditions. The gutter system simulates the overflow from an upstream node to a downstream inlet or channel as it would occur in the area being studied.

4.3 Peak Discharges

The peak discharges used in the Civil Storm models were obtained from the results of the "small scale" detailed hydrologic analyses and are provided in Appendix C - 9. Consistent with the scope of study, the discharges were modeled as peak flows, i.e. one-dimensional (1-D) flows for all of the systems. Civil Storm has functionality for both one-dimensional (1-D) and two-dimensional (2-D) calculation methods.

2-D modeling involves internals calculation of flows using defined hydrographs for the catchments associated with a particular feature such as a catch basin. Since the scope of study was developed to ratio the peak discharges from "large scale" hydrology, hydrograph data was not available for the HGL analysis. This approach was deemed conservative as 1-D modeling can account for attenuation and discharge reduction associated with hydrograph routing or reduction of flow in locations of significant storage.

4.4 Reach Boundary Conditions

The starting WSEs for most of the fifteen (15) areas were taken from the previously completed HEC-RAS modeling results. Areas 12, 14, and 15 were set to normal depth rather than having a known WSE, since Black Creek was not modeled in the open system analysis. Area 13's confluence is with Area 14, so its boundary conditions come from the water surface elevation determined during modeling Area 14.

WSEs from the 2-year storms were used for the 10-year/1-hour storms, since these storms were not modeled in HEC-RAS, but had similar flows in most cases. To determine the reach boundary conditions of each model the bounding cross sections at the confluence of the stream had to be determined. The water surface elevations at these two cross sections in the HEC-RAS model for the storm of interest were interpolated to determine the water surface elevation at the confluence. These reach boundary conditions can be seen in Appendix E - 1.

The use of tailwater conditions for the 10-year storm event is not required by Town design standards and therefore is considered conservative. In most study areas this tailwater condition had only a limited effect due to the constructed slope of the conveyance system and the natural terrain. In areas 3, 4, 8, and 10 the tailwater extended further into the system and created a flooded initial modeling condition. In these areas model runs were performed with and without tailwater to provide a full understanding of the limiting factors in the existing system. These additional runs were considered during development of master planning alternatives.

4.5 Cross Sections

Stream valley cross section data for the Civil Storm model came from a combination of the following sources:

- 2 foot contour data provided by the Town
- Field visits conducted by Dewberry during the open/closed system modeling portion of this project, including 60 relative-elevation GPS cross-sections
- Survey data of channel inverts taken from the Pilot and Maynard Loop Inventory

The field visit conducted during the open/closed system modeling utilized a global positioning system (GPS) unit for tracking stream locations where manual field measurements were conducted to determine the channels top width, bottom width, depth, and Manning's n value for an additional 62 cross sections. These cross sections were merged with topography data to create extended cross sections after the data was collected, which were drawn perpendicular to flow along the channel. Additional places where cross sections were typically included are upstream and downstream of road crossings and confluences, and at major changes in stream valley characteristics such as channel slope, roughness, or geometry. Once the locations of the cross sections were established, station-elevation data was collected from the topography to include the points into Civil Storm.

4.6 Manning's n Values

The hydraulic roughness coefficients (Manning's n-values) are a measure of the stream valley's resistance to flow, or in other words, the friction losses. Stream valleys with high roughness coefficients, for example channels with large boulders and densely forested overbanks, will tend to slow the flow and result in higher flood elevations, while relatively smooth areas, like mowed grass provide little resistance to flow and result in higher velocities and lower flood elevations.

Manning's n values were determined based off of aerials, photos, and stream meander. Values range from 0.02 to 0.078 and average 0.042. The high and low values in the range of Manning's n values used represent the extremes found in the TCAP area, where 0.02 represents water flowing on water in a pond or in some cases a clean concrete-lined channel, and 0.078 represents a channel with low flow with a rocky bottom and heavy brush on the immediate overbanks. Typical channel reaches in the TCAP area have bare soil or rip rap as a lining, and have light trees or brush on the overbanks, which were given a Manning's n value between 0.035 and 0.045 in most cases based off of values in the HEC-RAS user manual that were taken from Chow's 1959 "Open Channel Hydraulics".

4.7 Culverts

In urban areas, culvert crossings can often be the cause of flooding due to inadequate capacity. Therefore, it is important to properly model culverts so that their effects on flood elevations can be accurately determined.

The culvert data required for Civil Storm modeling includes:

- Inlet and outlet conditions
- Culvert type, material, shape, length, number of barrels, and dimensions
- Upstream and downstream invert elevations
- Top-of-road profile along the highest point on the road
- Entrance and Exit Loss Coefficients

This data was previously collected as part of the Town's Stormwater Inventory Database from the Maynard Loop and Pilot inventory studies, and was supplemented with additional survey as needed. Modeling culverts in Civil Storm (as discussed in section 4.10) required placing a manhole on the upstream and downstream end of a pipe modeled as a culvert. The rim elevations of the upstream and downstream manholes are set to be equivalent to the best known elevation of the road, and the diameter of the upstream manhole is the length that weir flow will likely occur during a flooding event.

4.8 Gutters

Gutters were used in the analysis to move overflow from a surcharging node to the next available downstream inlet. Gutters provide a method for modeling overland flow from one overtopping inlet to an inlet that can handle the inflow from the upstream inlet. The shape of a gutter typically simulated the dimensions of a pitched roadway with a 2% cross slope on either side of the center. Gutters are critical to maintaining the flow balance inside a system that experiences overflow, or the flow would not have a defined method of getting back into the system.

4.9 Summary of Hydraulic Results

Modeling results and level of service tables for each area studied can be viewed in Appendix F. Resulting hydraulic grade lines determined during modeling can be viewed in Appendix G. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in

Appendix G. General summary descriptions of each system and each system's modeled flooding problems are provided below. The referenced tables and HGL profiles are useful in visualizing the flood risks associated with the stormwater systems.

4.9.1 Area 1: Walnut Creek Basin

Evaluation of Area 1 indicates flooding concerns at the intersection of N. Lake St. and Pine St. and the adjacent area. The stormwater conveyance system in this area is a combination of pipes ranging in diameter from 18 to 48 inches and open channels that drain approximately 40 acres.

The modeled system consists of a single line that begins near the intersection of N. Lake St. and Pine St and continues downstream through Maple Avenue and finally discharging directly to Walnut Creek as shown in Appendix A. A number of residential structures can be found in close vicinity of the stormwater system.

The intersection at N. Lake St. and Pine St. is at risk of flooding in the 10-year/1-hour Existing and greater storm events. The modeling results indicate that there is a flooding risk to structures (748 E Chatham St and 607 Cedar St.) located around the north side of the pond immediately upstream of N. Lake St. Flooding in this location is affected by undersized pond outlet structure.

A summary of the results for Area 1 can be found in Appendix F - 1 and the resulting level of service for structures and roads along the system can be found in Appendix G - 1. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.2 Area 2: Walnut Creek Basin

The upstream extent of Area 2 is in the parking lot at 556 Chatham St. The system flows south until it discharges into Walnut Creek as shown in Appendix A. The modeling results indicate several areas are at risk for flooding, including is the parking lot of 556 Chatham St and 501 Waldo Street.

The results indicate the 556 Chatham Street potential flooding is caused by pipe 6020, which is undersized and creates a surcharge condition for the 10-year existing storm event and greater recurrence interval storms. 501 Waldo St. is also at risk from flooding during the 10-year existing event and greater recurrence interval storms. This drainage system in this area is a 15-inch pipe (pipe 0065) that receives inflow from a 24-inch pipe. Waldo St. is also at risk of flooding during all recurrence interval storms near pipe 0065.

A summary of the results for Area 2 can be found in Appendix F - 2 and the resulting level of service for structures and roads along the system can be found in Appendix G - 2. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.3 Area 3: Walnut Creek Basin

The analysis of Area 3 indicates several potential flood risk locations for the 10-year existing and future recurrence interval storms. Additionally, two areas also have a potential flood risk for all recurrence interval storms evaluated. The conveyance system begins approximately 650 feet west of Ward St. and continues downstream until it discharges into Walnut Creek as shown in Appendix A..

The most upstream modeled cross section and channel reach drain just over 13 acres. The structure at 311 Chatham St. is at risk for flooding at this location. The northwest corner of the Chatham St. and

Ward St. intersection is also floodprone for all recurrence interval storms. The flood risk at this location includes structures at 405 and 407 Chatham St.

The Chatham St. and Ward St. intersection is modeled to have flooding during the 10-year existing and 10-year future storm events due to surcharging parking lot inlets at 407 Chatham St. A flood risk exists for the 10-year existing storm and 10-year future storm events and is located at 414 Chatham St., which is a Town owned property. The houses at 412 and 418 Waldo St. are at risk during the 10-year existing and 10-year future storm events due to tailwater conditions from Walnut Creek. Additionally, Waldo St. shows a flood risk for the 10-year future event.

Flooding in this system is generally due to undersized pipes.

A summary of the results for Area 3 can be found in Appendix F - 3 and the resulting level of service for structures and roads along the system can be found in Appendix G - 3. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.4 Area 4: Walnut Creek Basin

The Area 4 drainage system starts upstream as a two branch drainage system. The two branches join just upstream of Walker Ave. One branch (nodes 8007 to 9073) drains north of Chatham St along Cedar St. and the other branch (nodes 8018 – 9073) drains south of Chatham St along Waldo St. After the two branches join the drainage system runs through a residential area to the southeast and continues until the confluence with Walnut Creek as shown in Appendix A.. The branch north (nodes 8007 to 9073) of Chatham St. has an upstream drainage area of approximately 1.6-ac. This area has flood risks associated with it for Cedar St. and some surrounding industrial buildings for the 10-year existing and greater recurrence interval storms. The downstream drainage area for this branch is approximately 10.1-ac.

The southern upstream branch (nodes 8018 – 9073) has an upstream drainage area of approximately 3.2-ac. This area has various flood risks associated with the 10-year future event, starting with a flood risk to Academy St. at the upstream end of the studied system. Waldo St. is also at risk of flooding during the 10-year future event. The 10-year existing event and greater recurrence interval storms pose a flood risk to the First United Methodist Church of Cary and surround properties. The downstream drainage area for this branch is approximately 12.4-ac.

The downstream portion of this drainage system has an upper end drainage area of approximately 21.2-ac and has several residential flooding risks associated with a variety of storms. The largest flood risk exists near the town homes near Stanopal Dr. and surrounding properties 222, 226, and 232 Waldo St for all studied recurrence interval storms. A culvert with a high head wall exists underneath the parking lot for the town homes and forces water to flow around the culvert into the backyards of residents living at 226 and 232 Waldo St. The water then overtops Stanopal Dr. and reenters the open channel near the town homes.

As the system continues downstream Hunter St. is put at flood risk during the 10-year existing and greater recurrence interval storm events. This risk is eliminated with the town's proposed changes that upgrade the culvert to a 2-ft by 6-ft box culvert. However, the flood risk to the surrounding town homes off Stanopal Dr. is not eliminated exists. Continuing downstream the 10-year existing storm event and greater recurrence interval storms produce flooding risks for the remainder of the system. The remaining system includes several residents off of Webster St. including the street itself. As the confluence with Walnut Creek is approached it becomes more difficult to associate the flooding risk with an undersized system. Rather it is the backwater from Walnut Creek that causes the flooding risk. However, the town's

proposed system does eliminate a property owner at 314 Webster St. from flood risk. The downstream drainage area is approximately 50.3-ac.

A summary of the results for Area 4 can be found in Appendix F - 4 and the resulting level of service for structures and roads along the system can be found in Appendix G - 4. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.5 Area 5: Walnut Creek Basin

The modeling results of Area 5 indicate flooding concerns at Walker St. and Fairview Rd. The conveyance system in this area is a combination of 24-inch pipes and open channel that drain approximately 29 acres, which begins near the intersection of Walker St. and Fairview Rd. and extends to Walnut St, and discharges into Walnut Creek as shown in Appendix A.. A number of commercial and residential structures can be found in close vicinity of the stormwater system.

The intersection at Walker St. and Fairview Rd. is at risk of flooding in the 10-year existing and future storm events. 320 S. Walker St. appears to be at risk of flooding during the 10-year existing and future storm events.

Flooding in these locations is affected by the headloss associated with the 24-inch culverts downstream of Walker St.

A summary of the results for Area 5 can be found in Appendix F - 5 and the resulting level of service for structures and roads along the system can be found in Appendix G - 5. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.6 Area 6: Swift Creek Basin

The modeling results of Area 6 indicate flooding concerns at South Harrison Avenue and Page St. and the structures in close proximity of the conveyance system. The stormwater conveyance system in this area is a combination of reinforced concrete pipes, ranging in diameter from 24 inches to 30 inches, and open channels that drain approximately 44 acres.

The system consists of a long reach of open channel at the upstream section followed by a piped section which outfalls to Swift Creek Tributary 7 as shown in Appendix A.. Two main streets cross this system, namely Page St. and S Harrison Avenue. A number of structures are in close vicinity to the stormwater system.

The S Harrison and Page St. intersection is overtopped by 10-year/1-hour Future and greater storm events. S Harrison is overtopped at two locations. The model results indicate flooding risk for the structures at 425 Harrison Avenue and 418 Page St for the 10-year existing and 10-year future storm events. The structure at 429 Harrison Avenue is at risk in the 10-year future event.

Flooding risk for the system is due to undersized pipe.

A summary of the results for Area 6 can be found in Appendix F - 6 and the resulting level of service for structures and roads along the system can be found in Appendix G - 6. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.7 Area 7: Swift Creek Basin

The modeling results for Area 7 indicate no flooding risk in the modeled portion of the stormwater system. The stormwater conveyance system begins upstream of Dry Avenue and extends to downstream of S. West St as shown in Appendix A.. The system in this area is a combination of pipes ranging in diameter from 24 to 36 inches and open channels that drain approximately 24 acres.

The system consists of single branch upto the intersection with S. West St. where the system divides into two branches allowing the overland discharge from the adjacent residential area to be introduced in the system. Both the branches drain into the Swift Creek Tributary 7 at a common outfall point. Various residential structures can be found in close vicinity of the stormwater system.

Model results indicate that there is no flooding risk to any structure or road along the system.

A summary of the results for Area 7 can be found in Appendix F - 7 and the resulting level of service for structures and roads along the system can be found in Appendix G - 7. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.8 Area 8: Swift Creek Basin

The modeling results for Area-8 indicate flooding risk for the majority of the system. The stormwater conveyance system in this area is a combination of pipes ranging in diameter from 18 to 48 inch circular pipes to a 2.6 by 2.4 box culvert and open channel that drain approximately 47 acres.

The system consists of one branch, which runs from just upstream of Harrison Avenue, north of W. Park Street, to its confluence with Swift Creek as shown in Appendix A.. Numerous residential structures are found in close vicinity of the stormwater system.

The pipe system upstream of Harrison Ave. is slightly undersized, and surcharging will likely only impact the parking lot upstream of Harrison Ave. The pipe system downstream of Harrison Ave. is undersized for the 10-year future storm event. The system has 30 inch pipe that flows to a 24-inch pipe near the intersection of Parkthrough and W. Park Street. Downstream of this location, the system has the risk of road flooding and structure flooding for the majority of the system length. The system experiences near the confluence with Swift Creek Tributary 7. However, downstream of Dixon St. the system is properly sized to handle the modeled storm events if backwater effects from Swift Creek Tributary 7 are not considered.

A summary of the results for Area 8 can be found in Appendix F - 8 and the resulting level of service for structures and roads along the system can be found in Appendix G - 8. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.9 Area 9: Swift Creek Basin

The modeling results for Area 9 indicate flooding risk along Old Apex Rd and on W. Chatham St. just after W. Chatham St. and Old Apex split. High House Rd and the nearby area also appear to experience flood risks. The stormwater conveyance system begins at the intersection of W Chatham St and S Dixon Ave. and runs along W Chatham St. to downstream of the intersection of W Chatham St and High House Rd as shown in Appendix A.. The stormwater conveyance system in this area is a combination of pipes ranging in diameter from 18 to 72 inches and open channel that drain approximately 63 acres.

The system consists of three branches; all three of which consist of a combination of pipes, manholes and inlets. Two smaller branches drain into the main pipe system which discharges into Swift Creek Tributary 7. Various commercial and residential structures can be found in close vicinity of the stormwater system. Old Apex Rd and W. Chatham St. just after W. Chatham St. and Old Apex split, are both at risk of flooding due to surcharging inlets during the 10-year existing and future flood events. The intersection of W. Chatham St. and High House Rd. is also at risk for flooding in the 10-year existing and future storm events. Additionally, there is flooding risk to a structure (809 W Chatham St) at the intersection of W Chatham St and High House Rd. Flooding in this location is affected by the headloss associated with the 4 ft box culverts at W Chatham St.

A summary of the results for Area 9 can be found in Appendix F - 9 and the resulting level of service for structures and roads along the system can be found in Appendix G - 9. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.10 Area 10: Coles Branch Basin

The modeling results for Area 10 indicate flooding risk at South Harrison Avenue and Page St. and the structures in close proximity of the system. The stormwater conveyance system in this area is a combination of reinforced concrete pipe, CMP, culverts, parallel pipes ranging in diameter from 15 inches to 30 inches, and open channel that drain approximately 25.1 acres.

The upstream portion of the system is a long reach of open channels and culverts followed by a parallel pipe system which outfalls to Coles Branch as shown in Appendix A.. CSX Transportation Rail Road, N West St., Holloway St., Wood St. and N Dixon Avenue either cross or run in parallel to the system. A number of structures can be found in close vicinity to the stormwater system.

N. West St. is overtopped by 10-year/1-hour future and larger storm events. The CSX Rail Road, Holloway St., Wood St are overtopped by the 2 hour future and 10-year storm events. N. Dixon Avenue is overtopped by all modeled storm events. N. Dixon is overtopped largely in part from tail water conditions of Coles Branch.

The model results indicate significant flooding risk to various structures (302 W Chatham St, 308 Chatham St. 318 W Chatham St., 102 N Dixon Avenue and 317 Holloway St.) for the 10-year/1-hour future and greater events,(202 N Dixon Av , 204 N Dixon Ave 315 Holloway St., 206 N Dixon Av and 310 N Dixon Av); for the 10-year/1-hour existing and greater events, (318 Holloway St., 322 Wood St, 320 Wood St. and 324 Wood St.); for the 2-year future and greater events, (206 N Dixon Av.); and for the 10-year existing and greater events.

Flooding risk is due to undersized pipes.

A summary of the results for Area 10 can be found in Appendix F - 10 and the resulting level of service for structures and roads along the system can be found in Appendix G - 10. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.11 Area 11: Coles Branch Basin

The modeling results of Area 11 indicate flooding risk at the NC/Norfork Southern Corporation railroad and the nearby area. The stormwater conveyance system begins at North Harrison Avenue and extends to South of Galveston Ct. and Middleton Avenue as shown in Appendix A.. The stormwater conveyance

system in this area is a combination of culverts ranging in diameter from 15 inches to 30 inches and open channel that drain approximately 35 acres.

The system is composed of two parallel pipe lines which tie to the main stem of Coles branch and a tributary that outfall to a common point. Various commercial structures can be found in close vicinity to the stormwater system.

The culvert crossing NC Railroad is overtopped by 10-year/1-hour future and greater storm events. The model results indicate that there is a flooding risk to a structure (214 Hillsboro St.) at the upstream of culvert. Flooding in this location is affected by the head loss associated with the 15-inch culvert under railroad.

A summary of the results for Area 11 can be found in Appendix F - 11 and the resulting level of service for structures and roads along the system can be found in Appendix G - 11. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.12 Area 12: Black Creek Basin

The modeling results for Area 12 indicate flooding risk at W. Johnson St. and Delta Ct. and the nearby area. The stormwater conveyance system in this area is a combination of pipes ranging in diameter from 18 to 42 inches and open channel that drain approximately 51 acres.

The system consists of two branches; one totally consists of open channels while other is a combination of pipes, manholes and inlets. The east branch of stormwater conveyance system begins at the intersection of W. Johnson St. and Delta Ct. and extends to west of Annagrey Dr. Both the branches drain into the Black Creek at a common outfall point as shown in Appendix A. Various commercial and residential structures can be found in close vicinity of the stormwater system.

The intersection at W. Johnson St. and Delta Ct. is at risk of flooding in the 10-year existing and 10-year future storm events. The model results indicate flooding risk to a structure (owned by: Northwoods Townhomes Association Limited) at the intersection of W. Johnson St. and Delta Ct.

Flooding in this location is related to the headloss associated with the 18-inch culverts downstream of W. Johnson St.

A summary of the results for Area 12 can be found in Appendix F - 12 and the resulting level of service for structures and roads along the system can be found in Appendix G - 12. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.13 Area 13: Black Creek Basin

Area 13 indicates flooding risks at E. Durham Rd. and at structures along Chapel Hill Rd., Johnson St., Bowden St., and Academy St. The studied stormwater conveyance system begins just south of NC-54/Chapel Hill Rd. near 316 Academy St., runs under E. Durham Rd., Chapel Hill Rd., and Johnson St., and then runs into its confluence just west of Gregory Dr as shown in Appendix A.. The stormwater conveyance system in this area is a combination of pipes ranging in diameter from 24 inches to 48 inches and open channel that drain approximately 75 acres, with an upstream drainage area of 7.4 acres.

The system is composed of a new pipe network on the upstream end at 316 Academy St, where a new parking deck has been constructed, and continues under Chapel Hill Rd. and Johnson St. through a commercialized area. Various commercial structures can be found in close vicinity to the stormwater system.

303 Bowden St. and E. Durham Rd. are expected to experience a flood risk during the 10-year/1-hour future and greater storm events. Flooding of 234 Johnson St. is anticipated to occur only in the 10-year future storm event. Based on the Town's existing topographic data, the new parking deck at 316 Academy St. is indicated to have flooding risk during the 10-year future storm event. The model results indicate the structures at 8330 and 8322 Chapel Hill Rd. are at risk in the 10-year future event.

Flooding in this system is likely due to an undersized pipe network between pipes 5146 and 5130.

A summary of the results for Area 13 can be found in Appendix F - 13 and the resulting level of service for structures and roads along the system can be found in Appendix G - 13. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.14 Area 14: Black Creek Basin

The modeling results for Area 14 indicate flooding risk just south of Chapel Hill Rd. and at Sorrell St. Several structures along this reach are also at flood risk. The studied stormwater conveyance system begins just south of Chapel Hill Rd. 1000 ft west of its intersection with Reedy Creek Rd., and ends just to the west of the intersection of Gregory Dr. and Branniff Dr as shown in Appendix A.. The stormwater conveyance system in this area is a combination of pipes ranging in diameter from 24 inches to 42 inches, a bridge with an approximate opening of 1.2 ft by 18 ft, and open channel that drain approximately 169 acres, with an upstream drainage area of 11.5 acres. 75 acres of drainage area enters the system on the downstream end to the west of 115 Gregory St.

The system is composed of mostly open channel, which is passed under crossings throughout the area through 5 culverts. A mixture of commercial and residential structures are found on the upstream end of this reach, while the downstream end of this reach is mostly surrounded by residential areas.

South of Chapel Hill Rd., the structures at 8075 and 8065 Chapel Hill Rd. are at risk for flooding during the 10-year existing and future flood events. This flooding risk is associated with the undersized culverts under the entrances to the parking lot. This portion of the system is also in backwater from the culvert running under Chapel Hill Rd. Sorrell St. is indicated to have flooding risk in the 10-year existing and future flood events, which places 101 Gregory Dr. in risk of flooding, as it will likely experience a flood risk during the 10-year/1-hour and greater storm events, regardless if the road overtops. Sorrell St. likely has an undersized culvert, and 101 Gregory St. may be located too close to the channel to avoid a flood risk in these flood events. Additionally, 119 Gregory St. is at flood risk in the 10-year/1-hour future, the 2-year future, and greater storm events, due to close proximity to the channel.

A summary of the results for Area 14 can be found in Appendix F - 14 and the resulting level of service for structures and roads along the system can be found in Appendix G - 14. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.9.15 Area 15: Black Creek Basin

The modeling results for Area 15 indicate flooding risk along N. Harrison Ave., W. Johnson St., and Gray St. and at a several adjacent structures. The studied stormwater conveyance system begins at the intersection of N. Harrison Ave. and Gray St. and extends just north of Queen Elizabeth Dr as shown in Appendix A.. The stormwater conveyance system in this area is a combination of pipes ranging in diameter from 15 inches to 48 inches and open channel that drain approximately 41 acres, with approximately 8.3 acres of drainage area on the upstream end of the system.

The system is composed of two parallel pipe lines on the upstream end, which converge at Harrison Ave. just north of Gray St, which then runs in a piped system to its outfall. Various residential structures can be found in close vicinity to the stormwater system.

Flooding is anticipated to occur in the 10-year existing and future storm events along, N. Harrison Ave. from its intersection with Gray St. to just north of W. Johnson St and south of E. Boundary St. This flooding is anticipated to cause an overflow over the curb near both 201 Johnson St. and 202 Gray St., which may flood the structures. It is likely, based on the hydraulic grade line in the upstream end of this system that the piped network upstream of Gray St. will also surcharge during the 10-year existing and future storm events. Flooding in this vicinity is due to an undersized pipe network.

A summary of the results for Area 15 can be found in Appendix F - 15 and the resulting level of service for structures and roads along the system can be found in Appendix G - 15. The HGL Index map provided in Appendix A can be utilized to determine the physical location of the HGL profiles found in Appendix G.

4.10 Additional Modeling Notes

Each modeled system had a unique set of requirements to properly model the system. Factors such as pipe length, parallel pipe networks, overflow, tailwater, inlet/outlet conditions, and others affect how a system must be modeled to produce balanced, expected results. Modelers should be aware that specific model upgrades were provided by Bentley during the development of this study. In order to obtain similar modeling results future users of the data will need to obtain the latest upgraded version of Civil Storm as of the printing of this report.

In cases where a manhole was overtopped it was not possible to add a gutter system to the manhole to carry the flow downstream. To maintain the overflow at manholes, so that it was not lost in the system; manholes had to be converted to inlets, which allowed for gutter connectivity.

In cases where culverts exist in the models, a manhole was placed on the upstream and downstream end of a pipe modeled as a culvert. The rim elevations of the upstream and downstream manholes are equivalent to the best known elevation of the road, and the diameter of the upstream manhole is the length that weir flow will likely occur during a flooding event.

Equivalent pipes were used in most locations where two parallel systems existed. This was done to assure that the results were balanced, as it is typically impractical to try to get two parallel pipe systems to give the exact same results at the upstream end where their HGLs should be identical. In cases where parallel pipes were used, the summary tables note this change.

Individual area modeling notes for each area have been summarized to inform potential reviewers or users of changes made.

Area 1

- Computation distance was changed to 1 ft.
- The pond is being modeled as a 'wide' channel.
- Pipe 6002 is modeled as a culvert.
- Manholes were added just upstream and downstream of culvert 6002 to model weir flow over the culvert.
- For modeling purpose, a very small culvert, 9345, was added downstream of the pond to model the riser box. Also, manholes were added just upstream and downstream of culvert 9345 to model weir flow over the culvert.
- The diameters for the manholes on the upstream ends of culverts 6002 and 9345 were set as the weir length to model the weir downstream of the pond.

Area 2

- Manhole 0066 was converted to an inlet to conserve overflow
- Cross section 9335 was added to the downstream channel reach
- Number of iterations and computation distance were set to 25 and 10-ft respectively.

Area 3

- The pipe network under and around 407 Chatham St. was modeled as equivalent pipes.
- Number of iterations and computation distance were set to 20 and 10-ft respectively.

Area 4 – Existing Conditions

- Downstream manholes 9264, 6155, & 9154 were converted to inlets to convey overflow further downstream.
- Manholes were placed upstream and downstream of culverts 9193, 9185, & 9184 so that weir flow could be calculated if the culvert was overtopped. Pipes 9055, 9057, & 9071 were modeled as equivalent pipes.
- Number of iterations and computation distance were set to 125 and 2-ft respectively.

Area 4 – Proposed Conditions

- Downstream manholes 9264, 6155, & 9154 were converted to inlets to convey overflow further downstream.
- Manholes were placed upstream and downstream of culverts 9193, 9185, & 9184 so that weir flow could be calculated if the culvert was overtopped. Pipes 9055, 9057, & 9071 were modeled as equivalent pipes.
- Number of iterations and computation distance were set to 75 and 3-ft respectively.

Area 5

- To conserve the overflow in the system, manholes 9244 and 9248) were changed to catch basins
- An inaccessible structure, manhole 9245 was removed to conserve the overflow in the system
- For modeling purpose, consecutive pipes 9171 and 9168 with diameters of 24-in and 30-in, respectively, are modeled as one 24" pipe

Area 6

- Default settings of computation distance and time step were changed to 10ft and 30.
- To conserve the overflow in the system all the manholes were changed to catch basin.

Area 7

- Computation distance and calculation time step were changed to 1 ft and 0.005 hr.

Area 8

- Computation distance was changed to 5 ft., and calculation options typically varied between runs
- Most downstream channels, 6005 and 6006 were removed from the model since they were overestimating the flow area. They were replaced with pipes CO-17 and 6111 with diameter 42" and 48", respectively
- The XSs u/s of both the channels were replaced with the catch basins.
- Manhole 8056 was changed to a catch basin to maintain continuity of flow, since the manhole was overflowing
- Catch basin was added at 8059 and 6089 to model overflow over Ridgecrest St. and Dry St., respectively
- The unverified structure node 6090 was removed from the model and a bend was put in the pipe in its place. Overflow was occurring at this node, but it was incorrect to show flooding of nearby structures when this cannot happen in the field
- Pipe 6111 was combined with parallel pipes 6113, 6112, and 6110 by making a box pipe with the equivalent area of the two pipe systems. A 42 in. and 48 in. pipe were turned into a 4 x 5.55 ft. box pipe

Area 9

- Computation distance and calculation time step were changed to 10 ft and 0.001 hr
- To conserve the overflow in the system manholes (DP 7056) was changed to catch basin
- Manholes were added just upstream and downstream of the culvert (DL 5164) to model weir flow over the culvert. In addition to manholes, channel and reaches were also included to maintain the connectivity of the system

Area 10

Due to the complexity of the system, the system was divided into three sections such as Upstream Middle and Downstream and modeled accordingly

- Upstream and Downstream Section: Computation distance and time step was changed to 10ft and 50
- Middle Section: Computation distance and time step was not changed.
- Manholes were added just upstream and downstream of the culvert to model weir flow over the culvert.
- Downstream section with parallel pipe system was modeled as an equivalent pipe system (where pipe sizes equivalent to the combined flow area of two parallel pipes)
- Pipe IO (cross-section just upstream of a pipe) was changed to a catch basin to model the overflow in the system.

Area 11

- Manholes were added just upstream and downstream of the culvert to model weir flow over the culvert. In addition to manholes, channel and reaches were also included to maintain the connectivity of the system.

Area 12

- Default setting for computation distance and time step was not changed.

Area 13

- Pipes 5142, 5137, 5136, and 5141 were converted to dual barrels the orientation of pipes 5141 and 5136 was used for modeling purposes. Two 30 in pipes were used to model this portion of the system because the 30-in pipes are the controlling factor for the flooding source in this area.
- Pipes 5199 and 5201(both 48-in pipes) were modeled as a dual barrel 48-in pipe
- Manholes 5134 and 5119 had to be converted to catch basins, since they were overflowing and had to allow gutter flow.
- Catch basin 5109 and pipe 5127 were removed and pipe 5128 was reconnected from catch basin 5110 to catch basin 5107. This was done to improve stability in the model in this area.
- Calculation options for the number of iterations were changed in this model due to split flow areas, which needed more iterations to balance the HGL. Computation distance and the calculation time step were also lowered.

Area 14

- Bridge 9484 had to be modeled as a culvert, since Civil Storm does not model bridges. Average dimensions of the bridge opening were used to model the bridge.
- Pipes 5114 and 5113 are being modeled as a culvert, rather than two separate pipes. This was useful in determining if Sorrell Street would experience flood risks.
- Manholes were used on the upstream and downstream end of all culverts.

Area 15

- Computation distance was changed to 5 feet.

4.11 Quality Control

In depth review of each model was completed to assure that inputs were correct and that the results were consistent with other methods. Review of the models included input data verification and flow regime comparisons performed by the modeler. This involved a check to

- verify that the model was consistent with the Town's Pilot and Maynard Loop Inventory
- confirm that pipes that were containing their flow had a consistent HGL as energy equation results
- validate culvert results from Civil Storm using Culvert Master for comparison

In locations where results varied the differences were noted, and the areas were investigated to confirm that Civil Storm and Culvert Master's results both showed a similar flood risk.

In addition to the modeler review a thorough review was performed by senior level personnel experienced in performing watershed planning and modeling. Consistency in modeling techniques and predictability of results were reviewed for each area modeled. Recommended changes were documented for implementation and corresponding results reviewed again to confirm outcome.

5.0 Conceptual Design Level Alternatives

The hydraulic grade line analysis modeling results were discussed with the Town to determine which areas required development of master plan Capital Improvement Project (CIP) alternatives. The Town's decisions on which areas required master plan CIP alternatives are shown in Table 18. Areas or segments of areas were excluded from the master plan development for several reasons, including:

- no flooding issues exist
- flooding issues characterized by privately owned systems
- the Town is already developing flood reduction strategies for these areas outside of this project

Table 18 Summary of Area Master Plan Response

Area	Master Plan Alternative Response
1	By Town's direction, no CIP developed. Flooding issues characterized by privately owned systems.
2	One (1) CIP alternative proposed.
3	Two (2) CIP alternatives proposed.
4	Three (3) CIP alternatives proposed. Downstream end has an ongoing improvement project, so no alternatives for this segment of the system were created.
5	One (1) CIP alternative proposed.
6	One (1) CIP alternative proposed.
7	No flooding issues identified.
8	Two (2) CIP alternatives proposed. Downstream end has an ongoing improvement project, so no alternatives for this segment of the system were created.
9	Three (3) CIP alternatives proposed.
10	Two (2) CIP alternatives proposed.
11	By Town's direction, no CIP developed. Flooding issues characterized by privately owned systems.
12	Two (2) CIP alternatives proposed.
13	Two (2) CIP alternatives proposed.
14	By Town's direction, no CIP developed. Flooding issues characterized by privately owned systems.
15	One (1) CIP alternative proposed.

5.1 General Approach

Flooding issues were evaluated on a system basis within each area. When evaluating flood reduction strategies for each area, the following information was utilized during the evaluation: 2 ft topography, color aerial photography (year 2000), stormwater infrastructure inventory data, and GIS layers for water distribution and sanitary sewer systems. Locations of other utilities (electrical, fiber optic, telephone, etc) were not known and could not be considered during this evaluation.

5.1.1 Development of Alternatives

The diversity of water quantity issues in the study area required evaluating a range of recommended alternatives for flooding prevention. Master plan alternatives range from pipe replacements/upgrades to more holistic stormwater system improvements.

Conceptual capital improvement design alternatives were developed and modeled in Civil Storm. Success criteria goals used to measure each proposed alternative included:

- Maintaining flow below crown of pipe for the 10-yr future conditions flow, when achievable
- Confirming that the conceptual design is physically feasible using available GIS and survey data (i.e. topography to confirm available pipe cover)
- Investigating methods to minimize sewer and water utility conflicts
- Minimizing easement acquisition
- Calculating economic feasibility of potential benefits

In some areas master planning alternatives that best met the above listed goals required extensive system improvements. In these cases, up to three (3) alternatives were provided that, to the best extent possible, utilized the existing stormwater system while maximizing the flood reduction benefit. These alternatives allow the HGL to exceed the crown of the pipe, but are anticipated to retain all flow within the stormwater system without surcharging during the 10-yr future condition.

Other elements that were considered in alternative selection included evaluating the ability to meet the desired level of service to roadways and structures and evaluating the level of permitting effort required. Permitting effort was evaluated using a three tier system of low, moderate, and high based on the perceived complicating permitting factors of the system. In general, a low level of effort rating is characteristic of a system with only pipe improvements. A moderate rating is characteristic of a system that is majority pipe improvements with small segments of open channel improvements. A high level of effort rating is characteristic of a system that has significant open channel reaches that need improvement or systems that utilized bypass pipes. Open channel segments increase the permitting effort due to the potential for required wetland mitigation, endangered species and stream buffer issues in certain watersheds.

5.1.2 Cost Estimation of Alternatives

Once alternatives were established for each flood risk area, planning level cost estimates were prepared for each alternative. A list of current unit prices for common stormwater improvement construction activities was developed from recent bids and industry references for use in the cost estimates. Anticipated construction cost was a key element in ranking the recommended flood reduction strategy. In developing the estimates of probable construction cost (EPCC), several assumptions were made, as follows:

- Mobilization was taken to be 10% of construction costs
- Construction stake out was assumed to be 5% of construction costs
- Contingency was set at 30% (10% for estimated engineering fees and 20% for project incidentals or construction details not included at the planning level)
- Easements would need to be obtained for all improved systems
- New junction boxes/inlets were included for each reach of storm sewer replaced

For easements, an average property value of $5 per square foot was utilized. This price was developed using average commercial and residential land tax values listed in the parcels layer within in the Town of Cary TCAP area. An easement width of 10-ft was assumed for pipes proposed to be constructed in the right-of-way, a width of 20-ft for pipes was assumed for pipe systems proposed to be constructed outside the right-of-way. Larger easements were assumed for proposed parallel systems and larger diameter pipes as necessary.

The preliminary EPCC is shown following each alternative discussion.

5.1.3 Utility Conflicts

Utility information available to assess conflicts was limited, and most conflicts had to be assumed based on GIS location data for water and sewer lines provided by the Town (no vertical information was available). When a water main or service line was crossed, vertical relocation for lines was assumed. Sewer crossing were generally assumed to not require vertical relocation unless additional vertical conflict could be inferred from the available data. Horizontal relocation of water and sewer lines was also considered when the proposed system was within 10-ft of existing lines. No specific cost estimate for other utilities (including gas, electrical, fiber optic, etc.) was included as these costs were assumed to be borne by the utility owner.

5.2 Alternatives Analysis and Recommendations

A brief summary of flooding issues and developed alternative(s) for each area are provided. Of the alternatives recommended, a preferred alternative was selected for each area based on flood reduction benefit, costs, and concerns voiced by the Town, if any.

5.2.1 Area 1 – No Master Planning (per the Town)

The upstream portion of this system has surcharging issues for all recurrence intervals. In addition an undersized pond outlet structure poses a flood risk to structures on the north side of the pond. At the Town's request no master planning was performed on Area 1 at this time. Flooding issues are largely characterized by privately owned systems. .

5.2.2 Area 7 – No Master Planning (per the Town)

Area 7 is a closed drainage system that outfalls to open channel flow at to the west of S West St. The only surcharging inlet in the system is located at 407 S West St and only surcharges during the 10-yr future storm event. After discussing this with the Town's staff it was decided that no master planning effort was desired.

5.2.3 Area 11 – No Master Planning (per the Town)

Area 11 is largely an open channel drainage system with piped flow underneath N Harrison Ave. and the railroad. The undersized culvert near the railroad poses a flooding risk for the building located at 214 Hillsboro St. After discussions with the Town's staff it was determine that no master planning was necessary at this time.

5.2.4 Area 14 – No Master Planning (per the Town)

Area 14 is a mostly open channel drainage system that utilizes culverts and small pipe networks to convey flow under roadways. Flood risks exist for structures at the upstream portion of the system (8075 & 8065 Chapel Hill Rd.) and for two structures along Gregory Dr. Flood risks are minor and are only reflected in the 10-yr existing storm event and greater. After discussions with the Town's staff it was determined that no master planning was desired at this time.

5.2.5 Area 2 – Master Planning Alternatives and Recommendation

Area 2 is a small, mostly open channel, drainage system. Flooding issues in this area are characterized by localized flooding sources. The flood problems in this area result from two undersized pipes: one located in on the upstream end of the system in the parking lot of 556 Chatham St. and the other in the middle of the system under Waldo St.

Evaluation of the system resulted in the development of one alternative. To remedy the flooding source at 556 Chatham St, it is recommended the existing 12-in RCP in the parking lot of 556 Chatham St. be replaced with a 15-in RCP and pipe slope modification. To remedy flooding at Waldo Street, it is recommended the existing 15-in RCP under Waldo St. be replaced with a 24-in RCP, while maintaining existing slope. Since the pipe under Waldo St. will maintain its current slope the existing water line in this vicinity was assumed to not be affected. The increased pipe size under Waldo St. will make it necessary to do minor channel improvements on the downstream end so that proper cover and outlet protection for the pipe can be achieved.

The recommended system upgrades are primarily pipe upgrades with minor channel improvements which means the permitting should expect a moderate level of effort. The two recommended pipe upgrades provide sufficient flow conveyance so that the flow will be contained below the crown of the pipes and all flood risks will be eliminated for the evaluated storms. The EPCC cost for the system upgrades is $65,000.

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 2 Proposed Master Plan Improvements
Alternative 1

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
12	15 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 56	18	$ 1,000
81	Catch Basins w/Frame and Grate	Each	$ 1,500	2	$ 3,000
80	4' Dia. Precast Manholes w/Frame and Grate (6 FT DEEP)	Each	$ 1,500	1	$ 1,500
98	24 inch Flared End Sections	Each	$ 600	1	$ 600
112	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	50	$ 2,000
10	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 98	146	$ 14,400
127	Asphalt or Bituminous Streets	SY	$ 40	107	$ 4,300
132	Sidewalk Replacement	SF	$ 4	160	$ 600
131	Curb and Gutter Replacement	LF	$ 20	40	$ 800

Estimate prepared by: Sheila Reeves, P.E.

Comments:

Note: Item 80: Modify inlet with 6" slot on US wall and add paved apron.

Subtotal Construction Cost	$ 28,000
Mobilization (10%)	$ 3,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 14,000
Contingency (30%)	$ 8,000
Property & Easements	$ 12,215
Total Estimated Cost	$ 65,000

5.2.6 Area 3 – Master Planning Alternatives and Recommendation

Area 3 is a mixed system that starts as open channel flow then is confined in pipe flow from 407 Chatham St. until the downstream end of Waldo St. An accessory building at 311 Chatham St. has a flood risk at the upstream portion of the system. The risk is minor and permitting and EPCC out weigh the potential benefit to avoid the minor flooding risk to the accessory building. The major flooding risk occurs at 405 and 407 Chatham St. The current split system that is under 407 Chatham St is inadequate to convey the flow coming from the upstream open channel portion of the system. Waldo St. is also a flood risk during the 10-yr future storm event due to backwater conditions from Walnut Creek. The Town instructed Dewberry to proceed with master planning for all areas of flooding in this system. Two (2) alternatives have been developed and evaluated for Area 3. Consideration of ground cover and the ability to tie into existing elevation control points were key factors that influenced master plan alternatives for this area. Alternative 1 concentrates on the master planning goal of maintaining pipe flow below the pipe crown, while alternative 2 maximizes the use of the existing infrastructure to minimize flooding risks. Each alternative is discussed below.

Alternative 1

The alternative in this case involves abandoning the drainage pipes directly under the building at 407 Chatham St. and upgrading the existing pipe system to the northeast of the building from (1) 24-in RCP to (3) 24-in RCP with a new slope. Pipes were limited to 24-in diameters because of existing ground cover in the area. This (3) 24-in pipe system will continue until just upstream of Ward St. where the system will increase to (2) new 30-in diameter pipes. The pipes that previously conveyed water under Ward St will be abandoned. The remainder of the system will be upgraded from a single 42-in RCP to (2) 36-in RCP pipes. A minor flood risk for the 10-yr future event will still exist at Waldo St. due to backwater from Walnut Creek. When the 10-yr tail water condition was changed to the 2-yr tail water condition the system was sufficient to convey the flow without surcharging and creating a flood risk for Waldo St.

Implementation of this alternative is physically achievable. Permitting is expected to be require a low level of effort given most of the improvements are recommended for the piped system. A complicating factor for this alternative is that it calls for improvements to every reach of the existing pipe in the system. Also, several sewer and water lines as well as Ward, Chatham, and Waldo Streets will be crossed to implement the alternative, which means the utility (water and sewer line) alterations could be difficult to achieve. The EPCC of this alternative is $985,000.

Alternative 2

Alternative 2 maximizes the existing storm system while balancing the master planning improvement success factors. While this alternative will not meet the goal of keeping flow below the crown of pipe, flow is kept below ground level. Similarly to Alternative 1 the drainage pipes directly under the building at 407 Chatham St. will be abandoned in place and a new headwall will be required at the upstream end where the open channel becomes pipe flow. The system to the northeast of the building at 407 Chatham St., that was converted to (3) 24-in pipes in Alternative 1, will remain in place and functioning. In fact, the current piped drainage system will remain unchanged until just upstream of the concrete pad in the urban park. This can be achieved by acquiring an easement from 405 Chatham St. and running (2) 24-in RCP pipes under Chatham St. to tie into an existing storm water drainage system branch (not included in this study). The branch that is tied into is assumed to need to be upgraded to (1) 36-in RCP. The four pipe segments from just upstream of the concrete pad in the urban park until just downstream of Waldo St. where the system outflows into open channel currently consist of (1) 42-in RCP pipe. The proposed alternative is to leave the current 42-in pipes in place and supplement them with a parallel 42-in pipe. This is sufficient to reduce the flood risk to Waldo St. up to the 10-yr future event. Although the pipes at the downstream end still do not flow below crown because of backwater conditions from Walnut Creek.

This alternative involves a bypass system and supplemental pipes on the downstream end of the system. As in Alternative 1, the permitting is assumed to require a low level of effort since the work is being done to a primarily piped system in an urban setting. Approximately the same number utilities (water and sewer) are affected for Alternative 2 as in Alternative 1. Therefore, utility problems may be difficult in either circumstance. Easements are also similarly priced between the two alternatives. The advantage of Alternative 2 is that the primary goal of flood risks reduction is acheived while utilizing a large portion of the existing storm system. The EPCC of this alternative is $685,000, which is $300,000 less than Alternative 1.

Recommended Alternative

Alternative 2 is the recommended alternative for this area.

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 3 Proposed Master Plan Improvements
Alternative 1

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
10	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 98	372	$ 36,600
104	Headwall/Endwall	CY	$ 500	7.0	$ 3,500
76	8' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 8,000	2	$ 16,000
77	7' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 6,000	6	$ 36,000
9	30 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 126	121	$ 15,300
8	36 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 151	1699	$ 256,900
83	Curb Inlet w/Hooded Grate	Each	$ 1,500	2	$ 3,000
84	Curb Inlet, Double, w/Frame and Grate	Each	$ 3,000	2	$ 6,000
139	10 inch Diameter and Smaller Water	Each	$ 5,000	4	$ 20,000
142	Water Relocation Along Length of Pipe	LF	$ 50	350	$ 17,500
136	15 inch Diameter and Smaller Sewer	LF	$ 45	230	$ 10,400
132	Curb and Gutter Replacement	LF	$ 20	120	$ 2,400
133	Sidewalk Replacement	SF	$ 4	100	$ 400
117	Clearing and Grubbing (Urban)	AC	$ 2,500	0.3	$ 700
128	Asphalt or Bituminous Streets	SY	$ 40	444	$ 17,800
113	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	111	$ 4,400
127	Concrete Court	SY	$ 34	600	$ 20,400

Estimate prepared by: Sheila Reeves, P.E.

Comments:

GIS shows multiple utility crossings. Water crossings include 2-8" crossings, 2-2" crossings, 1-6" pipe that will have to be relocated along a 350 LF length .

Note:

Sewer crossings include 1-24" RCP sewer, 4-8" VCP crossings and 230LF of 8" VCP to be relocated.

Subtotal Construction Cost	$ 467,000
Mobilization (10%)	$ 47,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 234,000
Contingency (30%)	$ 140,000
Property & Easements	$ 96,900
Total Estimated Cost	$ 985,000

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination
Area 3 Proposed Master Plan Improvements
Alternative 2

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
136	15 inch Diameter and Smaller Sewer	LF	$ 45	454	$ 20,400
104	Headwall/Endwall	CY	$ 500	0.9	$ 400
10	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 98	300	$ 29,500
8	36 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 151	259	$ 39,200
78	6' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 4,500	1	$ 4,500
83	Curb Inlet w/Hooded Grate	Each	$ 1,500	3	$ 4,500
133	Sidewalk Replacement	SF	$ 4	1300	$ 5,200
132	Curb and Gutter Replacement	LF	$ 20	322	$ 6,400
139	10 inch Diameter and Smaller Water	Each	$ 5,000	5	$ 25,000
7	42 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 196	436	$ 85,300
128	Asphalt or Bituminous Streets	SY	$ 40	459	$ 18,400
75	10' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 12,500	4	$ 50,000
127	Concrete Court	SY	$ 34	600	$ 20,400

Estimate prepared by: Sheila Reeves, P.E.

Comments:

GIS shows multiple utiltity crossings. Water crossings include 2-8" crossings, 2-2" crossings, 1-6" pipe that will have to be relocated along a 350 LF length .

Note:

Sewer crossings include 1-24" RCP sewer, 4-8" VCP crossings and 230LF of 8" VCP to be relocated.

Subtotal Construction Cost	$ 309,000
Mobilization (10%)	$ 31,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 155,000
Contingency (30%)	$ 93,000
Property & Easements	$ 96,900
Total Estimated Cost	$ 685,000

5.2.7 Area 4 – Master Planning Alternatives and Recommendation

The upstream portion of this area is drained by a branched drainage system consisting of mostly pipe flow until the two branches combine at S. Walker St. A single pipe network then continues until 216 Waldo St. where the flow becomes open channel, with the exception of three culverts, until the downstream portion of the system at Hunter St where the drainage system becomes piped again. The major flooding problems occur at the Church on Academy St., along Cedar St, near the town homes along Stanopal Dr., and along Webster St. After discussions with the Town's staff it became apparent that the Town was already formulating ways to resolve the flooding issues with the Church on Academy St. The Town has also contracted an engineering consultant to design a new pipe network for the downstream portion of the model running from just upstream of Hunter St until the outfall into Walnut Creek. It is our understanding that the goal of the downstream design was to provide a 10-yr level of service for Hunter St. This condition was modeled in the existing conditions hydraulic grade line modeling and indicated that although the roadway level of service was achieved several homes along Webster St. as well as the street itself are still at flood risk. Therefore, the alternatives for Area 4 do not completely resolve the flooding issues associated with the Church on Academy St., the townhomes on Stanopal Dr., or the homes along Webster St. Three alternatives were developed for Area 4. The upstream portion of the system is the same in all alternatives. This means that permitting for this portion of the system should require a low level of effort for all alternatives due to the fact that this is an urban setting with a piped drainage system. The variations in master planning alternatives occur at the downstream end of the model around the townhomes off Stanopal Dr.

Alternative 1

The master planning alternative for the upstream portion of this area focuses on the northern drainage branch that runs parallel to Ceder St. The pipe segment adjacent to 111 Chatham St. needs to be upgraded from the existing 15-in RCP to an 18-in RCP. The remaining pipes downstream of this point until the small open channel segment adjacent to 149 Chatham St. need to be upgraded from existing 15-in RCP to 24-in RCP. The small open channel segment will then be piped to provide a continuous closed drainage system for the northern branch of the upstream system. The small reach of 24-in diameter pipe at 158 Chatham St, just before the system has a minor split, currently has an inverse slope. In this alternative the inverts of the pipe have been adjusted to better convey flow. The final upgrade on the upstream portion of the model occurs on a pipe segment that runs parallel to S Walker St. adjacent to 127 Waldo St. A proposed 36-in RCP would replace the current 30-in RCP. The middle portion of the system needs channel improvements at the southwestern corner of 216 Waldo St. The channel improvements would more than double the capacity of the current channel and would increase the slope associated with the channel. In turn, this would lower the upstream invert associated with the culvert directly behind the house located at 216 Waldo St. The alternative, in addition to adjusting the slope to the culvert behind 216 Waldo St., calls for an upgrade of the existing 36-in CMP culvert to a 2.5-ft by 5-ft box culvert. The final improvement for Alternative 1 involves the culvert that runs underneath the parking lot for the townhomes off Stanopal Dr. The existing (2) 24-in RCP are to be upgraded with (2) 2-ft by 4-ft box culverts. This improvement merely reduces the flood risk to the surrounding townhomes, while not eliminating it. The Town's planned downstream improvements limit the further effectiveness of additional flood reduction strategies without drastically reworking the downstream portion of the system. This will be discussed in Alternatives 2 and 3 descriptions.

Although this alternative does not eliminate all of the flooding issues, it does meet the goal of keeping flow below the crown of the pipes for the northern branch of the upstream portion of the system and reduces the flood risk associated with the property surrounding Stanopal Dr. by approximately 0.5-ft to 0.2-ft in the 10-yr future event. The utility interference is minimal in this alternative and permitting

should be moderate for all alternatives. The EPCC for Alternative 1 at $645,000 is significantly lower than Alternatives 2 and 3 while providing marginally higher flood risks.

Alternative 2

Alternative 2 utilizes all of the upgrades from Alternative one and then adds a bypass system that parallels Stanopal Dr., Crosses Hunter Street and then parallels Webster St. until out-falling into Walnut Creek just downstream of Webster St. The most upstream pipe in the bypass system is a 2-ft by 5-ft box culvert due to ground cover in the area. After the initial pipe (2) 36-in RCP are proposed to the outfall at Walnut Creek. The alignment of the proposed bypass system can be seen in the Alternative 2 "Downstream Proposed Improvements" map attached in this section.

Alternative 2 provides all the benefits of Alternative 1 along with additional flood reduction risk of 0.7-ft for the properties surrounding Stanopal Dr. However, a flood risk still exists in this area and with the proposed improvements utilities and easement acquisition become significantly more of an issue. The EPCC for Alternative 2 is $1,625,000 and almost $1,000,000 more than that of Alternative 1.

Alternative 3

Alternative 3 utilizes all of the upgrades from Alternative one except for the culvert upgrade under the parking lot at Stanopal Dr. In Alternative 3 the culvert under the parking lot is plugged and a new headwall is added so that a bypass system can divert the water around the townhomes off of Stanopal Dr. The bypass system is similar to that of Alternative 2 except that it ties into the Town's proposed improvements just downstream of Hunter St. The alignment of the bypass for Alternative 3 can be seen in the Alternative 3 "Downstream Proposed Improvements" map attached in this section. The effectiveness of the bypass is limited by the downstream proposed pipe network design and provides flood reductions in a similar range to that of Alternative 2.

Alternative 3 provides all the benefits of Alternative 1 with similar flood reduction risks as Alternative 2 for properties surrounding Stanopal Dr. Utility crossings and easement acquisition are slightly less of an issue, but still difficult, for Alternative 3 when compared to Alternative 2. The EPCC for Alternative 3 is $1,130,00, which is approximately $485,000 more than Alternative 1 and approximately $495,000 less than Alternative 2.

Recommended Alternative

Alternative 1 is the recommended alternative for area 4. This alternative addresses the flooding issues that the Town of Cary is not currently addressing with the exception of Webster St. which is currently being upgraded by a different engineering consulting firm with another set of modeling objectives. Other alternatives will provide further flood risk reduction, but not eliminate flooding in the area surround Stanopal Dr. and Webster St. Spending the amount of money necessary to implement Alternative 2 or 3 does not appear to be justified considering the marginal additional flood risk reduction. If an alternative that eliminates flood risks in the downstream area is desired then it is recommended that further investigation takes place.

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 4 Proposed Master Plan Improvements
Alternative 1

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
11	18 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 71	158	$ 11,200
10	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 98	485	$ 47,700
80	4' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 1,500	9	$ 13,500
128	Asphalt or Bituminous Streets	SY	$ 40	1138	$ 45,500
132	Curb and Gutter Replacement	LF	$ 20	592	$ 11,800
142	Water Relocation Along Length of Pipe	LF	$ 50	359	$ 18,000
	2.5 FT X 5 FT PRECAST BOX CULVERT (6" WALLS)	LF	$ 504	60	$ 30,200
74	2' x 4' - PRECAST CONCRETE BOX CULVERT	LF	$ 520	107	$ 55,600
139	10 inch Diameter and Smaller Water	Each	$ 5,000	1	$ 5,000
	Channel Improvement	LF	$ 150	47	$ 7,100
104	Headwall/Endwall	CY	$ 500	0.8	$ 400
8	36 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 151	98	$ 14,800

Estimate prepared by: Sheila Reeves, P.E.

Comments:

GIS indicates a 6" water line located within R/W of Cedar St. A 6" water main crossing on Chatham St. A 6" sanitary sewer crossing is shown at culvert #9185.

Note:

Subtotal Construction Cost	$ 261,000
Mobilization (10%)	$ 26,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 131,000
Contingency (30%)	$ 78,000
Property & Easements	$ 150,800
Total Estimated Cost	$ 647,000

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 4 Proposed Master Plan Improvements
Alternative 2

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
11	18 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 71	158	$ 11,200
10	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 98	485	$ 47,700
80	4' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 1,500	9	$ 13,500
128	Asphalt or Bituminous Streets	SY	$ 40	2523	$ 100,900
132	Curb and Gutter Replacement	LF	$ 20	1410	$ 28,200
142	Water Relocation Along Length of Pipe	LF	$ 50	495	$ 24,800
	2.5 FT X 5 FT PRECAST CONCRETE BOX CULVERT (6" WALLS)	LF	$ 504	60	$ 30,200
74	2' x 4' - PRECAST CONCRETE BOX CULVERT	LF	$ 520	107	$ 55,600
139	10 inch Diameter and Smaller Water	Each	$ 5,000	3	$ 15,000
136	15 inch Diameter and Smaller Sewer	LF	$ 45	567	$ 25,500
104	Headwall/Endwall	CY	$ 500	1.6	$ 800
76	8' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 8,000	8	$ 64,000
8	36 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 151	1624	$ 245,500
	2 ft x 5 ft PRECAST BOX CULVERT	LF	$ 530	132	$ 70,000
	Channel Improvement	LF	$ 47	47	$ 2,200

Estimate prepared by: Sheila Reeves, P.E.

Comments:

GIS indicates a 6" water line located within R/W of Cedar St and Stanopal St. A 6" water main crossing on Chatham St and three (3) additional water main crossings. A 6" sanitary sewer crossing is shown at culvert #9185 and at three other locations along improvements.

Note:

Subtotal Construction Cost	$ 735,000
Mobilization (10%)	$ 74,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 368,000
Contingency (30%)	$ 221,000
Property & Easements	$ 226,675
Total Estimated Cost	$ 1,625,000

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 4 Proposed Master Plan Improvements
Alternative 3

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
11	18 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 71	158	$ 11,200
10	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 98	485	$ 47,700
80	4' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 1,500	9	$ 13,500
128	Asphalt or Bituminous Streets	SY	$ 40	1609	$ 64,400
132	Curb and Gutter Replacement	LF	$ 20	848	$ 17,000
142	Water Relocation Along Length of Pipe	LF	$ 50	495	$ 24,800
	2.5 FT X 5 FT PRECAST CONCRETE BOX CULVERT (6" WALLS)	LF	$ 504	60	$ 30,200
74	2' x 4' - PRECAST CONCRETE BOX CULVERT	LF	$ 520	107	$ 55,600
139	10 inch Diameter and Smaller Water	Each	$ 5,000	4	$ 20,000
8	36 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 151	578	$ 87,400
	2 ft x 5 ft PRECAST CONCRETE BOX CULVERT	LF	$ 504	132	$ 66,500
136	15 inch Diameter and Smaller Sewer	LF	$ 45	401.0	$ 18,000
76	8' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 8,000	5	$ 40,000
104	Headwall/Endwall	CY	$ 500	0.8	$ 400
	Channel Improvement	LF	$ 47	47	$ 2,200

Estimate prepared by: Sheila Reeves, P.E.

Comments:

GIS indicates a 6" water line located within R/W of Cedar St and Stanopal St. A 6" water main crossing on Chatham St and three (3) additional water main crossings. A 6" sanitary sewer crossing is shown at culvert #9185 and at three other locations along improvements.

Note:

Subtotal Construction Cost	$ 499,000
Mobilization (10%)	$ 50,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 250,000
Contingency (30%)	$ 150,000
Property & Easements	$ 180,850
Total Estimated Cost	$ 1,130,000

5.2.8 Area 5 – Master Planning Alternatives and Recommendation

Area 5 is a small mixed drainage system that consists of an open channel upstream drainage system that flows into a piped system to cross S Walker St. and Fairview Rd. The system returns to open channel flow on the east side of S Walker St. on the backyard property line of 325 S Walker St. until it confluences with Walnut Creek. There are a couple of flood risks associated with the Area 5 drainage system. The only structural flood risk occurs at the upstream end of the system at 320 S Walker St. and is due to an inadequate pipe slope. Roadway flood risks exist at S Walker St. and Fairview Rd. due to undersized pipes. After discussions with the Town's staff it was decided to look at the system to investigate possible improvements. After investigation one Alternative is suggested.

This master planning alternative involves providing a new headwall and adjusting the slope for the most upstream pipe in the system. With the exception of the pipe that runs parallel to S Walker St. on the west side, the remaining pipes in the system need to be upgraded from the existing 24-RCP to 30-in RCP. In addition a new outlet on the downstream pipe as well as outlet protection will be required. The water and sewer utilities that could possibly be displaced are minor and the permitting effort for the suggested upgrades should be minimal considering the pipe network is in an urban setting. Easements are not a major concern for this area because they represent less than 10% of the final EPCC. The EPCC for this alternative is $167,000 and meets the objective of keeping flow below the crown of the pipes as well as achieving the Town's goal of flood risks reduction.

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 5 Proposed Master Plan Improvements
Alternative 1

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
9	30 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 126	224	$ 28,200
83	Curb Inlet w/Hooded Grate	Each	$ 1,500	3	$ 4,500
79	5' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 2,800	2	$ 5,600
97	30 inch Flared End Sections	Each	$ 900	1	$ 900
113	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	267	$ 10,700
139	10 inch Diameter and Smaller Water	Each	$ 5,000	1	$ 5,000
132	Curb and Gutter Replacement	LF	$ 20	260	$ 5,200
128	Asphalt or Bituminous Streets	SY	$ 40	220	$ 8,800
10	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 98	12	$ 1,200
104	Headwall/Endwall	CY	$ 500	1	$ 500
142	Water Relocation (10" and smaller)	LF	$ 50	120	$ 6,000
84	Curb Inlet, Double, w/Frame and Grate	Each	$ 3,000	1	$ 3,000

Estimate prepared by: Sheila Reeves, P.E.

Comments:

Note: Utility Relocations (along Walker St). GIS indicates 8" sewer crossing and 6" water main crossing re-alignment. (Assumed design will avoid sewer conflict)

Subtotal Construction Cost	$ 80,000
Mobilization (10%)	$ 8,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 40,000
Contingency (30%)	$ 24,000
Property & Easements	$ 14,900
Total Estimated Cost	$ 167,000

5.2.9 Area 6 – Master Planning Alternatives and Recommendation

Area 6 starts as an open channel drainage system at 601 Kildare Farm Rd and becomes pipe flow just upstream of S Harrison Ave at 429 S Harrison Ave. until it outfalls back to an open channel downstream (West) of Page St. The system surcharges at several locations through out the system causing flood risks for S Harrison Ave. and Page St. Structural flood risks exist for 418 Page St., 425 S Harrison Ave and 429 S Harrison Ave. After discussions with the Town's Staff utilities appeared as if they may be difficult to deal with. However, the town still wanted to investigate possible flood reduction strategies. After investigation one Alternative is suggested.

The master planning alternative starts with channel upgrades on the upstream portion of the system for approximately 325-ft of stream in order to improve the conveyance of the channel and to appropriately transition from open channel flow to pipe flow. All pipes in the pipe network for this area are to be upgraded. Currently pipes are either 24-in or 30-in RCP. The most upstream pipe segment, running under S Harrison Ave., is to be replaced with a 2.16-ft by 3-ft concrete box. The next two downstream pipe segments are to be replaced with a 2-ft by 4-ft concrete box. The remaining pipes are to be upgraded to (2) 3-ft by 3-ft concrete boxes. It should be noted that if these improvements are made improvements to the downstream open channel and culverts (not studied) would most likely be required to ensure the downstream capacity. The possible downstream channel improvements were not part of this study and the EPCC is estimated based on the required channel cross section at the end of the studied reach. The channel and downstream culverts are part of the HEC-RAS model and will require further evaluation to identify appropriate improvements. This Alternative keeps the pipe flow at or below the crown of pipe for the entire pipe network and reduces all flood risks in the surrounding area that was studied.

There are several water and sewer utility lines in the vicinity of this drainage system. The improvements suggested could displace some lines and may prove difficult for the Town. Permitting effort may be extensive due to the significant length of open channel improvements suggested. The cost for the channel improvement noted in the EPCC does not cover land cost and conservation easement cost. This is a jurisdictional stream and may not qualify for credit without a home buyout. With the assumptions noted the total EPCC for this Alternative is approximately $773,000.

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 6 Proposed Master Plan Improvements
Alternative 1**

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
113	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	67	$ 2,700
81	Catch Basins w/Frame and Grate	Each	$ 1,500	3	$ 4,500
78	6' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 4,500	6	$ 27,000
128	Asphalt or Bituminous Streets	SY	$ 40	320	$ 12,800
139	10 inch Diameter and Smaller Water	Each	$ 5,000	2	$ 10,000
	Precast 26" x 36" Box Culvert	LF	$ 354	51	$ 18,100
	Precast 3' x 3' Box Culvert	LF	$ 374	376	$ 140,600
	Precast 2 ft x 2 ft Box Culvert	LF	$ 266	286	$ 76,100
104	Concrete Headwall/Wall	CY	$ 500	1.2	$ 600
132	Curb and Gutter Replacement	LF	$ 20	238	$ 4,800
133	Sidewalk Replacement	SF	$ 4	80	$ 300
117	Clearing and Grubbing (Urban)	AC	$ 2,500	0.1	$ 300
	Channel Improvement*	LF	$ 300	325	$ 97,500

Estimate prepared by: Sheila Reeves, P.E.

Comments: Assume sanitary sewer will not be a conflict. Water Crossings include 6" and 2" lines.

Note:

*Assume stream restoration techniques to achieve conveyance improvements. Does not include land cost and conservation easement cost. May not qualify for credit without home buyout. Stream appears on the USGS map and is jurisdictional. - no permitting fees specifically included in estimate.
**Improvements to the stormwater system will be needed downstream of the master plan study area. Costs does not include improvements required downstream of master plan area.

Subtotal Construction Cost	$ 395,000
Mobilization (10%)	$ 40,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 198,000
Contingency (30%)	$ 119,000
Property & Easements	$ 20,500
Total Estimated Cost	$ 773,000

5.2.10 Area 8 – Master Planning Alternatives and Recommendation

Area 8 is a mixed stormwater drainage system. The studied system starts at 116 W Park St., just east of S Harrison Ave., and drains to the southwest until just west of S Dixon Ave at 324 S Dixon Ave. The majority of the system is undersized and will need to be upgraded in order to pass the 10-yr future storm event. Currently the majority of the properties and roadways along this drainage system experience flood risks for at least the 10-yr existing storm event. After discussions with the Town's staff it was realized that the Town is already dealing with flooding issues at the downstream portion of the system near 339 Dry Ave. The Town of Cary has preliminary plans to buy the house located at 339 Dry Ave. and upgrade pipes in order to alleviate flooding risks in this area. Therefore, the master planning efforts were focused from just downstream of Ridgecrest Rd. to the upstream study limit to the east of S Harrison Ave. Two different Alternatives were developed for this area. The Alternatives involve upgrading pipe sizes, converting pipe systems to open channel, or converting open channel systems to pipe systems. Only two alternatives will be presented, but a combination of the two alternatives is possible depending on the preference of the Town.

Alternative 1

Existing pipe sizes from the upstream study limit to just upstream of S Harrison Ave. are 18-in diameter. Alternative 1 calls for upgrading existing pipes in this vicinity to a single 24-in RCP. The small pipe segment downstream to the east of S Harrison Ave. needs to be upgraded from an existing 24-in RCP to a 30-in RCP. The current open channel that separates properties at 208 and 122 S Harrison Ave. will require significant improvements in order to carry the 10-yr existing storm event and a lower downstream invert so that the channel can flow directly into a pipe inlet rather than the current yard inlet. A gabion wall will be required for a large portion of the proposed open channel because of the proposed lower invert of the channel bed in combination with a narrow area for channel improvements. The pipes that run from the newly proposed gabion wall channel outfall downstream until 311 W Park St. all need to have their inverts adjusted and pipe sizes increased. Pipe sizes in this area vary from24-in to 30-in RCP. The upgrade calls for (2) 30-in RCP for this entire reach of pipe network. Further system improvements are suggested for the pipe network that runs under the structure located at 318 Ridgecrest Rd. Alternative 1 proposes buying the structure at 318 Ridgecrest Rd. and converting the current piped system into and open channel system until the culvert that runs under Willow St. The current pipe under Willow St. is a 24-in RCP and would need to be upgraded to a 2.5-ft by 5-ft box culvert in order to convey the flow from the 10-yr future storm event. The final system upgrades for Area 8 suggested for Alternative 1 include upgrading the small pipe network that conveys flow under Ridgecrest Rd. Currently the pipe network consists of three pipe segments. The two most upstream pipe segments are 30-in RCP and the most downstream pipe in this pipe network is a 2.6-ft by 2.4-ft box. In order to pass the 10-yr future storm event the existing pipes need to be upgraded to a 2.5-ft by 5-ft concrete box. As previously mentioned further downstream system improvements area already being investigated by the Town.

There are several sewer and water utility lines in this area. The proposed changes call for increased pipe sizes and new inverts so utility conflicts are likely and may be difficult to work around. Permitting could also be more difficult due to improving one channel reach and creating a new open channel segment. This Alternative also involves purchasing a currently occupied structure. The EPCC for Alternative 1 is $1,850,000.

Alternative 2

Alternative 2 utilizes all of the pipe upgrades from Alternative 1. The major difference between the two alternatives is that Alternative 2 calls for piping two existing open channel segments and avoiding the structure purchase proposed in Alternative 1. The current open channel, that is considered the property

boundary between 208 and 122 S Harrison Ave., is to be piped with (2) 30-in RCP. The other proposed flood reduction strategy for Alternative 2 involves piping the open channel segment just upstream of the structure at 318 Ridgecrest Rd. The current pipe network underneath the structure would be abandoned in place and a pipe bypass system would divert the flow around the structure within the right-of-way of Ridgecrest Rd. The new bypass pipe system would tie into the pipe that runs under Willow St.

The pipe bypass system that parallels Ridgecrest Rd. will encounter more utility problems than the suggested open channel from Alternative 1, but purchase of the structure would not be necessary to implement the improvement for Alternative 2. Permitting effort should be similar to Alternative 1. Easement and property acquisitions are the same for Alternatives 1 and 2, other than the purchase of the structure at 318 Ridgecrest Rd. Both alternatives also meet the level of service goal and reduce flood risks in the area. The EPCC for Alternative 2 is $1,678,000.

Recommended Alternative

Alternative 2 is the recommended alternative for area 8. Alternative 2 costs $200,000 less than Alternative 1 and avoids the property buy-out and the construction of an open channel. It should be noted that a combination of the two alternatives may also be possible at a similar cost if the Town desires to pursue other alternatives.

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 8 Proposed Master Plan Improvements
Alternative 1

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
74	2.5' x 5' - PRECAST CONCRETE BOX CULVERT (6" WALLS)	LF	$ 504	133	$ 67,000
106	Headwall/Endwall	CY	$ 500	4.6	$ 2,300
131	Asphalt or Bituminous Streets	SY	$ 40	1222	$ 48,900
135	Curb and Gutter Replacement	LF	$ 20	480	$ 9,600
142	10 inch Diameter and Smaller Water	Each	$ 5,000	7	$ 35,000
139	15 inch Diameter and Smaller Sewer	LF	$ 45	1125	$ 50,600
114	Excavate Drainage Trench	LF	$ 11	87	$ 1,000
	Purchase property at 318 Ridgecrest Dr.	Each	$ 209,684	1	$ 209,700
	House Demolition	Each	$ 5,300	1	$ 5,300
	Pipe Disposal	LF	$ 25	180	$ 4,500
9	30 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 126	1489	$ 187,600
99	30 inch Flared End Sections	Each	$ 900	3	$ 2,700
136	Sidewalk Replacement	SF	$ 4	125	$ 500
	Gabion wall (3 ft base, 5 ft high)	LF	$ 300	275	$ 82,500
10	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 98	187	$ 18,400
82	4' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 1,500	1	$ 1,500
85	Curb Inlet w/Hooded Grate	Each	$ 1,500	3	$ 4,500
87	Yard Inlets, Side Inlet w/Flat Slab Top	Each	$ 2,000	1	$ 2,000
116	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	33	$ 1,300
79	7' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 6,000	16	$ 96,000
143	12 inch Diameter and Larger Water	Each	$ 7,500	1	$ 7,500

Estimate prepared by: Sheila Reeves, P.E.

Comments: GIS indicates numerous water main crossings ranging in size from 2" up to 12" main. Numerous sanitary sewer crossings noted as well. Assumed conflicts with sewer would be prevalent because pipe size & slope are changing with proposed changes.

Note:

Subtotal Construction Cost	$ 838,000
Mobilization (10%)	$ 84,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 419,000
Contingency (30%)	$ 251,000
Property & Easements	$ 258,100
Total Estimated Cost	$ 1,850,000

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination
Area 8 Proposed Master Plan Improvements
Alternative 2

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
74	2.5' x 5' - PRECAST CONCRETE BOX CULVERT (6" WALLS)	LF	$ 504	82	$ 41,300
106	Headwall/Endwall	CY	$ 500	3.4	$ 1,700
131	Asphalt or Bituminous Streets	SY	$ 40	2003	$ 80,100
135	Curb and Gutter Replacement	LF	$ 20	733	$ 14,700
142	10 inch Diameter and Smaller Water	Each	$ 5,000	7	$ 35,000
139	15 inch Diameter and Smaller Sewer	LF	$ 45	838	$ 37,700
79	7' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 6,000	21	$ 126,000
9	30 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 126	2854	$ 359,600
99	30 inch Flared End Sections	Each	$ 900	3	$ 2,700
10	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 98	187	$ 18,400
85	Curb Inlet w/Hooded Grate	Each	$ 1,500	4	$ 6,000
87	Yard Inlets, Side Inlet w/Flat Slab Top	Each	$ 2,000	1	$ 2,000
116	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	33	$ 1,300
	Fill	CY	$ 7	508	$ 3,600
136	Sidewalk Replacement	SF	$ 4	125	$ 500
143	12 inch Diameter and Larger Water	Each	$ 7,500	1	$ 7,500
87	Yard Inlets, Side Inlet w/Flat Slab Top	Each	$ 2,000	1	$ 2,000
10	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 98	46	$ 4,500
	2" water line	LF	$ 6	324	$ 2,000

Estimate prepared by: Sheila Reeves, P.E.

Comments: GIS indicates numerous water main crossings ranging in size from 2" up to 12" main. Numerous sanitary sewer crossings noted as well. Assumed conflicts with sewer would be prevalent because pipe size & slope are changing with proposed changes.

Note:

Subtotal Construction Cost	$ 747,000
Mobilization (10%)	$ 75,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 374,000
Contingency (30%)	$ 224,000
Property & Easements	$ 258,100
Total Estimated Cost	$ 1,678,000

5.2.11 Area 9 – Master Planning Alternatives and Recommendation

Area 9 is a branched urban drainage system with mixed conveyance that drains from just east of S Dixon Ave. to 809 Chatham St. The majority of the flood risks in this area are associated with roadways (S Dixon Ave., W Chatham St, and Old Apex Road). There is one structural flood risk at 809 Chatham St. After discussion with the Town's staff it was determined that problems in the system be evaluated and addressed with a master planning alternative. The majority of the alternatives involve upgrading pipes. However, Alternative 3 does include a bypass system instead of major pipe upgrades.

Alternative 1

The first pipe upgrade for Alternative 1 is implemented just downstream of the upstream W Chatham St. crossing (pipe ID 7079). The existing 24-in RCP is to be replaced with a 30-in RCP. The next downstream pipe segment will have the same pipe upgrade (pipe ID 7078). The next two downstream pipes, on the main drainage branch, are currently 42-in diameter pipes. The current size of these pipes is adequate, but in order to meet the objective of maintaining flow at or below the crown of pipes the slope needs to be adjusted. The slope adjustment of these two pipes makes it necessary to adjust the slope of the 42-in pipe that runs underneath/near 602 W Chatham St. Rather than trying to upgrade the pipe near the building at 602 W Chatham St. the alignment has been shifted to avoid the structure. This allows pipe slope adjustment without interfering with the existing structure. A new manhole was required in order to realign the single pipe segment into two pipe segments that flow around the building. The two pipe segments downstream of 602 W Chatham St. exist as single 36-in RCP. It is proposed that they are upgraded to single 42-in pipes. The most downstream pipe in the upstream drainage system of Area 9 is to upgraded from the existing 42-in pipe to (2) 36-in RCP that have flared end sections and outlet protection. The final upgrade in Alternative 1 involves adding a second 4-ft by 4-ft box culvert underneath W Chatham St. at the downstream end of the system. Although the culvert upgrade does not allow flow to be below the crown of the pipe approximately 2-ft of freeboard is acheived between the water surface elevation of the 10-yr future storm event and the top of roadway elevation.

The rest of the system improvements allow the goal of keeping flow below the crown of pipes to be met. There is an 8-in water main that will be crossed during the suggested improvements, but this is required for all alternatives. Permitting effort should be less considering the only improvements are a pipe and culvert upgrades in an urban setting. Easements are just a small fraction of the costs for this alternative. The overall EPCC for Alternative 1 is $507,000.

Alternative 2

Alternative 2 is similar to that of Alternative 1 in that only pipe upgrades are suggested to reduce the flooding risks in the area. Alternative 2 utilizes the same pipe upgrades as Alternative 1 for the pipes that are recommended for upgrade (pipe ID's 7079, 7078, 7056, and 5164). However, the pipe that runs underneath 602 W Chatham St. is not to be upgraded. The two pipe segments upstream (pipe ID's 7070 and 7069) and the two pipe segments downstream (pipe ID's 7066 and 7065) of 602 W Chatham St. will also remain unchanged. Alternative 2 does not meet the goal of keeping all the flow below the crown of pipes, but involves significantly less pipe upgrades and construction work reducing cost while still reducing all flood risks from studied storms to structures and roadways in this area.

Permitting should be less complicated than in Alternative 1 since less ground disturbing activities are proposed to implement the improvements. The utility interference will be the same as in Alternative 1 and involve crossing an 8-in water main. Easements are a small fraction of the cost for this alternative as in Alternative 1. The overall EPCC for Alternative 2 is $282,000, which is $225,000 less than Alternative 1.

Alternative 3

Alternative 3 utilizes a bypass pipe network in order to divert flow around the middle portion of the upstream drainage system (the portion of the pipe network between Old Apex Rd. and W Chatham St.). The upstream pipe that crosses underneath W Chatham St. will be plugged or diverted by a weir in the newly proposed junction box to facilitate the bypass pipe network that will continue to parallel W Chatham St. until just north of W Park St. where W Chatham St. will then be crossed. The bypass system will join the existing drainage system where the upstream pipe network discharges into open channel flow at 606 W Chatham St. The proposed bypass pipe alignment can be seen in the attached figure for Area 9 labeled "Area 9 – Alternative 3 Upstream Proposed Improvements". The only other recommended improvement for this alternative deals with the downstream culvert that runs underneath W Chatham St. An additional 4-ft by 4-ft box culvert will be added to supplement the current 4-ft by 4-ft box culvert.

Easement acquisition will be greater with Alternative 3 than in Alternatives 1 and 2. Permitting will also be more difficult for Alternative 3 since a pipe system will be laid in a new alignment. There will also be more conflict with water and sewer utilities for this Alternative when compared to other Alternatives. Overall the Alternative 3 meets the goal of maintaining flow below the crown of pipes. The EPCC for Alternative 3 is $439,000.

Recommended Alternative

Alternative 2 is the recommended alternative for this area. Although Alternative 2 does not meet the goal of keeping flow below the crown of pipes for all pipe segments, it does minimize the flood risk in the area for all studied storms. Alternative 2 is also $157,000 less than the next least expensive alternative. Alternative 2 utilizes the largest portion of existing infrastructure when comparing alternatives which means permitting and construction should require less effort than other alternatives. Alternative 2 has less utility impacts than Alternative 3 and similar utility impacts to Alternative 1.

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 9 Proposed Master Plan Improvements
Alternative 1

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
73	4 FT X 4 FT BOX CULVERT	LF	$ 376	54	$ 20,300
128	Asphalt or Bituminous Streets	SY	$ 40	1200	$ 48,000
113	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	139	$ 5,600
109	Channel Slope Grading	LF	$ 5	100	$ 500
8	36 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 151	280	$ 42,300
96	36 inch Flared End Sections	Each	$ 1,200	2	$ 2,400
7	42 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 196	404	$ 79,000
79	5' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 2,800	4	$ 11,200
83	Curb Inlet w/Hooded Grate	Each	$ 1,500	5	$ 7,500
9	30 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 126	252	$ 31,800
132	Curb and Gutter Replacement	LF	$ 20	195	$ 3,900
133	Sidewalk Replacement	SF	$ 4	820	$ 3,300
139	10 inch Diameter and Smaller Water	Each	$ 5,000	1.0	$ 5,000
104	Headwall/Endwall	CY	$ 500	1.5	$ 700

Estimate prepared by: Sheila Reeves, P.E.

Comments:

GIS shows 2" force main crossing and an 8" water main crossing. Assume force main will not conflict.

Note:

Subtotal Construction Cost	$ 262,000
Mobilization (10%)	$ 26,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 131,000
Contingency (30%)	$ 79,000
Property & Easements	$ 8,600
Total Estimated Cost	$ 507,000

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination
Area 9 Proposed Master Plan Improvements
Alternative 2

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
73	4 FT X 4 FT BOX CULVERT	LF	$ 376	54	$ 20,300
128	Asphalt or Bituminous Streets	SY	$ 40	612	$ 24,500
8	36 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 151	280	$ 42,300
96	36 inch Flared End Sections	Each	$ 1,200	2	$ 2,400
9	30 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 126	252	$ 31,800
83	Curb Inlet w/Hooded Grate	Each	$ 1,500	3	$ 4,500
79	5' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 2,800	1	$ 2,800
132	Curb and Gutter Replacement	LF	$ 20	177	$ 3,500
133	Sidewalk Replacement	SF	$ 4	160	$ 600
113	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	139	$ 5,600
109	Channel Slope Grading	LF	$ 5	100	$ 500
139	10 inch Diameter and Smaller Water	Each	$ 5,000	1.0	$ 5,000

Estimate prepared by: Sheila Reeves, P.E.

Comments:

GIS shows 2" force main crossing and an 8" water main crossing. Assume force main will not conflict.

Note:

Subtotal Construction Cost	$ 144,000
Mobilization (10%)	$ 14,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 72,000
Contingency (30%)	$ 43,000
Property & Easements	$ 8,600
Total Estimated Cost	$ 282,000

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 9 Proposed Master Plan Improvements
Alternative 3

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
73	4 FT X 4 FT BOX CULVERT	LF	$ 376	54	$ 20,300
128	Asphalt or Bituminous Streets	SY	$ 40	780	$ 31,200
9	30 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 126	782	$ 98,500
80	4' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 1,500	4	$ 6,000
83	Curb Inlet w/Hooded Grate	Each	$ 1,500	1	$ 1,500
133	Sidewalk Replacement	SF	$ 4	2720	$ 10,900
139	10 inch Diameter and Smaller Water	Each	$ 5,000	1	$ 5,000
136	15 inch Diameter and Smaller Sewer	LF	$ 45	549	$ 24,700
118	Clearing and Grubbing (Open)	AC	$ 5,000	0.1	$ 500
113	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	139	$ 5,600
104	Headwall/Endwall	CY	$ 500	0.7	$ 300

Estimate prepared by: Sheila Reeves, P.E.

Comments:

GIS indicates 8" water main crossing, 8" sanitary sewer crossing

Note:

Subtotal Construction Cost	$ 205,000
Mobilization (10%)	$ 21,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 103,000
Contingency (30%)	$ 62,000
Property & Easements	$ 48,200
Total Estimated Cost	$ 439,000

5.2.12 Area 10 – Master Planning Alternatives and Recommendation

Area 10 is a largely open channel drainage system that starts just west of S. Harrison Ave. The open channel system parallels the railroad until just before N. Dixon Ave. where the channel turns north and uses a series of culverts and pipe networks to flow underneath property at 317 Holloway St., Holloway St., Wood St., and property at 310 N Dixon Ave. Existing conditions hydraulic grade line modeling indicates that a large majority of the pipes in the system are undersized. The undersized pipes create flooding risks for the railroad, Holloway St. and surrounding properties, Wood St. and surrounding properties, and N. Dixon Ave and surrounding properties. It should be noted that tail water conditions for the Coles Branch open channel modeling cause the downstream pipes to flow full in the studied storm events and that all alternatives should also consider the additional costs of the required downstream improvements to the culvert under N. Dixon Avenue. After discussions with the Town's staff it was determined that master planning efforts should focus on eliminating flood risks where feasible. The exception to this is the railroad crossing, Evaluation of the system after discussions with the Town resulted in two (2) alternatives. Each alternative shares the same improvements for the downstream portion of the model. The major differences occur in the middle portion of the model. The differences are illustrated in the attached "Area 10 – Alternative 1/2 Upstream Proposed Improvements" maps.

Alternative 1

Increasing the existing culvert under N West St. from a single 18-in RCP to (3) 18-in RCP addresses the flood risk associated with N. West St. Alternative 1 calls for upgrading the culvert underneath the property at 317 Holloway St. Increasing the number of barrels and upgrading the pipe size would be difficult at this location without realigning the pipe system due to the proximity of the structure. The pipe network that conveys water under Holloway St. will need to be upgraded to single barrel 30-in RCP to eliminate the flood risk for Holloway St. Further downstream the ditch will need to be cleaned and sangged to remove heavy deposition and dense vegetation. The downstream pipe network that runs across the property located at 310 N. Dixon Ave. will need to be upgraded from (1) 24-in RCP and (1) 30-in RCP to (1) 2.5-ft by 5-ft concrete box.

This alternative does not meet the goal of keeping flow below the crown of pipes, but does achieve reduced flood risks to structures and roadways. The tail water conditions from Coles Branch along with limited ground cover make it difficult to meet conveyance goals in this area. Permitting for this alternative will require a moderate effort considering a major pipe upgrade will occur in a resident's back yard and minor channel modification will be required in order to implement other culvert upgrades. Easements estimated at over 20% of the projects cost are a significant issue to deal with. The numerous sewer and water utility lines in the area have the potential to complicate the system. Utilities could require a high level of effort in order to implement the suggested alternative. The EPCC for this alternative is $590,000.

Alternative 2

Alternative 2 proposes increasing the existing 18-in RCP under N. West St. to (3) 18-in RCP. Just upstream of the culvert located on 317 Holloway St. a new head wall and bypass pipe system will be installed. The alignment of this new system can be seen on the attached map labeled as "Area 10 – Alternative 2 Upstream Proposed Improvements". The bypass system will include three segments of pipe that are (2) 1.17-ft by 1.9-ft elliptical concrete pipe and one 30-in RCP pipe segment. The culvert that drains under or near the structure located on property at 317 Holloway St. will be left open. The culvert will then be connected to the proposed bypass system at a newly proposed manhole. The bypass system will tie into the existing pipe network just upstream of Holloway St. where the remaining pipes will be upgraded to 30-in RCP. The downstream system improvements mimic those of Alternative 1.

Alternative 2 does not meet the goal of keeping flow below the crown of pipes, but does address the majority of the flooding risks. A minimal flood risk still exists for properties upstream of the railroad as in Alternative 1. The bypass system for Area 10 utilizes property lines for alignment and may be less complicated to implement than Alternative 1 that calls for construction in a resident's backyard. Since the bypass system will require more pipe easement acquisition cost is slightly greater than in Alternative 1. Permitting will require moderate effort as in Alternative 1 due to minor channel modifications and increasing pipe sizes. Utilities have the potential to be difficult to deal with as in Alternative 1. The EPCC for this alternative is $604,000.

Recommended Alternative

Alternative 2 is the recommended alternative for this area. The deciding factor was the constructability of the project. Both alternatives have similar costs and both meet the level of service goal set out by the Town's staff within the improved area. However, Alternative 1 calls for construction close the structure located on 317 Holloway St. where Alternative 2 utilizes property lines to realign the pipe system. The downstream improvements are the same for Alternatives 1 and 2.

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 10 Proposed Master Plan Improvements
Alternative 1

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
104	Headwall/Endwall	CY	$ 500	1.3	$ 600
74	2.5' x 5' - PRECAST CONCRETE BOX CULVERT	LF	$ 342	223	$ 76,100
83	Curb Inlet w/Hooded Grate	Each	$ 1,500	8	$ 12,000
81	Catch Basins w/Frame and Grate	Each	$ 1,500	1	$ 1,500
111	Existing Ditch Cleaning (Minor Clearing)	LF	$ 7	50	$ 400
97	30 inch Flared End Sections	Each	$ 900	4	$ 3,600
9	30 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 126	248	$ 31,200
113	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	200	$ 8,000
11	18 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 71	500	$ 35,400
99	18 inch Flared End Sections	Each	$ 500	14	$ 7,000
153	Fencing - Chain link	LF	$ 20	40	$ 800
132	Curb and Gutter Replacement	LF	$ 20	280	$ 5,600
128	Asphalt or Bituminous Streets	SY	$ 40	382.2	$ 15,300
136	15 inch Diameter and Smaller Sewer	LF	$ 45	565	$ 25,400
139	10 inch Diameter and Smaller Water	Each	$ 5,000	3	$ 15,000

Estimate prepared by: Sheila Reeves, P.E.

Comments:

Pipe just downstream of Civil Storm model has negative slope (is inlcuded in RAS model). Improvements will likely need to be made downstream of Civil Storm Model to have an effective storm flow improvement.

Note:

RAILROAD CROSSING IS UNDERSIZED AND CAUSES BACKWATER. ONE PROPERTY MAY BE IMPACTED BY BACKWATER. NO IMPROVEMENTS FOR RR CULVERT PROPOSED. GIS INDICATES SEVERAL WATER AND SS CROSSINGS.

Subtotal Construction Cost	$ 238,000
Mobilization (10%)	$ 24,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 119,000
Contingency (30%)	$ 71,000
Property & Easements	$ 138,100
Total Estimated Cost	$ 590,000

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination
Area 10 Proposed Master Plan Improvements
Alternative 2

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
104	Headwall/Endwall	CY	$ 500	1.3	$ 600
74	2.5' x 5' - PRECAST CONCRETE BOX CULVERT	LF	$ 342	223	$ 76,100
83	Curb Inlet w/Hooded Grate	Each	$ 1,500	8	$ 12,000
81	Catch Basins w/Frame and Grate	Each	$ 1,500	1	$ 1,500
111	Existing Ditch Cleaning (Minor Clearing)	LF	$ 7	50	$ 400
97	30 inch Flared End Sections	Each	$ 900	4	$ 3,600
9	30 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 126	299	$ 37,700
113	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	200	$ 8,000
	1.17 FT X 1.92 CORRUGATED ELLIPTICAL PIPE	LF	$ 57	386	$ 22,000
80	4' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 1,500	3	$ 4,500
132	Curb and Gutter Replacement	LF	$ 20	280	$ 5,600
128	Asphalt or Bituminous Streets	SY	$ 40	382.2	$ 15,300
136	15 inch Diameter and Smaller Sewer	LF	$ 45	565	$ 25,400
139	10 inch Diameter and Smaller Water	Each	$ 5,000	3	$ 15,000
153	Fencing - Chain link	LF	$ 20	40	$ 800
11	18 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 71	80	$ 5,700

Estimate prepared by: Sheila Reeves, P.E.

Comments:

Pipe just downstream of Civil Storm model has negative slope (is inlcuded in RAS model). Improvements will likely need to be made downstream of Civil Storm Model to have an effective storm flow improvement.

Note:

RAILROAD CROSSING IS UNDERSIZED AND CAUSES BACKWATER. ONE PROPERTY MAY BE IMPACTED BY THE BACKWATER. NO IMPROVEMENTS FOR RR CULVERT PROPOSED. GIS INDICATES SEVERAL WATER AND SS CROSSINGS.

Subtotal Construction Cost	$ 234,000
Mobilization (10%)	$ 23,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 117,000
Contingency (30%)	$ 70,000
Property & Easements	$ 160,150
Total Estimated Cost	$ 604,000

5.2.13 Area 12 – Master Planning Alternatives and Recommendation

Area 12 is a two branch system. The west branch is entirely open channel and the east branch starts as primarily a pipe system that flows into an open channel then back into a pipe network at the downstream end. The majority of the studied system is sized properly and the only structural flood risk associated with this area occurs at the upstream end of the system near the Northwood Townhomes Association. After discussions with the Town's staff it was determined that master planning was only needed for the upstream portion of the drainage system. Two alternatives were developed for this area.

Alternative 1

Alternative 1 involves upgrading the two most upstream studied pipe segments on the east branch of the drainage system from existing 18-in RCP to 24-in RCP. This adresses flooding risks in the system. However, the goal of keeping flow below the crown of pipe is not met in this alternative. The goal can be met by adjusting the proposed diameter of the pipe underneath the parking lot to an elliptical pipe with an equivalent area to a 30-in RCP. However, elliptical RCP can be costly for small application and matching conveyance with a single elliptical CMP may not be possible due to cover constraints.

Since the inverts on all pipe upgrades are being held utility interference is expected to be minimal. Permitting should require a low level of effort because upgrades only involve increases in pipe sizes. The EPCC of this alternative is $107,000.

Alternative 2

Alternative 2 suggests upgrading the two most upstream studied pipe segments on the east branch of the drainage system from exiting 18-in RCP to 24-in RCP, like in Alternative 1. The replacement of the small open channel portion and a small 24-in RCP pipe just downstream of the Northwood Townhomes Association with a 24-in RCP allows the system to meet the conveyance goal of maintaining flow below the crown of pipes.

This alternative will require more extensive permitting than Alternative 1 due to a more elaborate master planning improvements. There is no additional flood reduction benefit for the studied storms in this area when compared to Alternative 1. No additional utilities are encountered with Alternative 2, which means water and sewer utilities should still require minimal effort to deal with. The EPCC of this alternative is $122,000, which is $15,000 (~14% greater in price) more than Alternative 1.

Recommended Alternative

Alternative 1 is the suggested alternative for this area. Alternative 1 utilizes more of the existing infrastructure than Alternative 2. The result is that Alternative 1 costs $15,000 less than Alternative 2. Piping the small section of open channel that Alternative 2 suggests provides no further flood reduction benefit and results in more effort during permitting and construction.

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 12 Proposed Master Plan Improvements
Alternative 1

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
10	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 98	274	$ 27,000
83	Curb Inlet w/Hooded Grate	Each	$ 1,500	2	$ 3,000
98	24 inch Flared End Sections	Each	$ 600	1	$ 600
131	Curb and Gutter Replacement	LF	$ 20	60	$ 1,200
127	Asphalt or Bituminous Streets	SY	$ 40	422	$ 16,900
112	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	37	$ 1,500
		Subtotal Construction Cost			$ 50,000
		Mobilization (10%)			$ 5,000
		Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)			$ 25,000
		Contingency (30%)			$ 15,000
		Property & Easements			$ 12,215
		Total Estimated Cost			$ 107,000

Estimate prepared by: Sheila Reeves, P.E.

Comments:

Note: Item 80: Modify inlet with 6" slot on US wall and add paved apron.

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 12 Proposed Master Plan Improvements
Alternative 2

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
10	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 98	321	$ 31,600
83	Curb Inlet w/Hooded Grate	Each	$ 1,500	2	$ 3,000
80	4' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 1,500	2	$ 3,000
131	Curb and Gutter Replacement	LF	$ 20	60	$ 1,200
127	Asphalt or Bituminous Streets	SY	$ 40	422	$ 16,900
		Subtotal Construction Cost			$ 56,000
		Mobilization (10%)			$ 6,000
		Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)			$ 28,000
		Contingency (30%)			$ 17,000
		Property & Easements			$ 15,215
		Total Estimated Cost			$ 122,000

Estimate prepared by: Sheila Reeves, P.E.

Comments:

Note: Item 80: Modify inlet with 6" slot on US wall and add paved apron.

5.2.14 Area 13 – Master Planning Alternatives and Recommendation

The drainage system for Area 13 at the upstream study limits starts as a closed pipe network then becomes open channel flow for approximately 360-ft before it becomes a closed system again. There is a small section of open channel adjacent to 303 Bowden St. before the pipe system continues downstream to the outfall at the northern end of a large undeveloped parcel at 101 Sorrell St. Several flood risks in this system are a result of an under sized pipe system. After discussions with the Town's staff it was decided to only look at the downstream portion of the system. The upstream portion of the system surcharging appears to be caused by the downstream conveyance constraints. The Town wanted to see an alternative that involved piping the small open channel segment adjacent to 303 Bowden St. and an alternative that involved leaving the small open channel section alone. Essentially the two alternatives developed are the same otherwise.

Alternative 1

Alternative 1 flood reduction measures start just downstream of E Durham Rd. The existing 36-in pipe that runs across the property at 303 Bowden St. will be upgraded to (2) 48-in RCP. The small open channel segment just downstream will be left untouched other than new inlet protection for the proposed pipe upgrade under Chapel Hill Rd. The existing (2) 30-in RCP are to be replaced with (2) 48-in RCP. The pipe segments downstream of this point until just upstream of E. Johnson St. consist of a single 48-in CMP. This alternative calls for the additional of another 48-in CMP or RCP pipe to supplement the existing pipe until just upstream of E. Johnson St. At this point the new supplemental pipe will join the inlet on the south side of E. Johnson St. and the small pipe segment (pipe ID 5128) just upstream of E. Johnson St. will be removed. From E. Johnson St. until the downstream limit of study the system is sized appropriately as (2) 48-in CMP.

This alternative meets the service level goal of maintaining flow below the crown of pipes for the studied storms while minimizing cost. All flood risks were addressed with Alternative 1. Permitting efforts for this area will be low due to the fact that all improvements are increasing pipe sizes in an urban environment. There are sewer and water utilities in the area. A 6-in and a 20-in water main run down Chapel Hill Rd. as well as an 8-in VCP sanitary sewer. In order to deal with the utilities in the area a moderate level of planning may be required. Easements acquisition will be the same for both alternatives. The EPCC of this alternative is $867,000.

Alternative 2

Alternative 2 utilizes the same pipe improvements as Alternative 1. In Alternative 2 the short open channel section at 303 Bowden St. was piped with (2) 48-in RCP.

This alternative meets the service level goal of maintaining flow at or below the crown of pipes for the studied storms. All flood risks were addressed with Alternative 2. Permitting efforts for this alternative would require more effort from that of Alternative 1 due to piping and open channel segment. Sewer and water utility interference as well as easement acquisition will be the same for both alternatives. The EPCC for Alternative 2 is $920,000, which is $53,000 more than Alternative 1.

Recommended Alternative

The recommended alternative for this area is Alternative 1. Similarly to Area 12 the variation between Alternative 1 and Alternative 2 in this area is piping a small open channel segment or leaving it as open channel. Leaving the small open channel segment adjacent to 303 Bowden St. alone is a less expensive alternative that results in the same reduction in flood risk. Permitting and construction efforts should also be less considering all recommended improvements from Alternative 1 are pipe improvements.

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination
Area 13 Proposed Master Plan Improvements
Alternative 1

Probable Construction Cost Estimate

Dewberry

April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
75	10' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 12,500	1	$ 12,500
32	48 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (8 - 10 FT DEEP)	LF	$ 266	915	$ 243,800
94	48 inch Flared End Sections	Each	$ 1,800	4	$ 7,200
113	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	444	$ 17,800
79	5' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 2,800	4	$ 11,200
139	10 inch Diameter and Smaller Water	Each	$ 5,000	1	$ 5,000
140	12 inch Diameter and Larger Water	Each	$ 7,500	1	$ 7,500
83	Curb Inlet w/Hooded Grate	Each	$ 1,500	1	$ 1,500
128	Asphalt or Bituminous Streets	SY	$ 40	1553	$ 62,100
132	Curb and Gutter Replacement	LF	$ 20	160	$ 3,200

Estimate prepared by: Sheila Reeves, P.E.

Comments:

Pipe section #5133 and #5130 will utilize existing 48" CMP pipe. Alt #1 proposes to add a parallel 48" RCP barrel to system. Has been modeled as 2 - CMP's to be conservative.

Note:

GIS indicates a 6" and 20" water mains running down Chapel Hill Rd. Also, an 8" VCP sanitary sewer is shown to cross storm along Chapel Hill Rd. Assume SS will not conflict.

Subtotal Construction Cost	$ 372,000
Mobilization (10%)	$ 37,000
Engineering,, Const. Staking, As-Built, and Misc. Proffessional Services (50%)	$ 186,000
Contingency (30%)	$ 112,000
Property & Easements	$ 160,000
Total Estimated Cost	$ 867,000

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 13 Proposed Master Plan Improvements
Alternative 2

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
75	10' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 12,500	1	$ 12,500
32	48 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (8 - 10 FT DEEP)	LF	$ 266	1015	$ 270,400
113	Rip Rap and/or Rock Channel Protection w/ Filter Fabric	SY	$ 40	444	$ 17,800
79	5' Dia. Precast Manholes w/Frame and Grate (6FT DEEP)	Each	$ 2,800	6	$ 16,800
139	10 inch Diameter and Smaller Water	Each	$ 5,000	1	$ 5,000
140	12 inch Diameter and Larger Water	Each	$ 7,500	1	$ 7,500
83	Curb Inlet w/Hooded Grate	Each	$ 1,500	1	$ 1,500
128	Asphalt or Bituminous Streets	SY	$ 40	1553	$ 62,100
132	Curb and Gutter Replacement	LF	$ 20	160	$ 3,200
108	Fill Short Channel Section	CY	$ 10	333	$ 3,300

Estimate prepared by: Sheila Reeves, P.E.

Comments:

Pipe section #5133 and #5130 will utilize existing 48" CMP pipe. Alt #2 proposes to add a parallel 48" RCP barrel to system and enclose short open channel segment. Has been modeled as 2 - CMP's to be conservative.

Note:

GIS indicates a 6" and 20" water mains running down Chapel Hill Rd. Also, an 8" VCP sanitary sewer is shown to cross storm along Chapel Hill Rd. Assume SS will not conflict.

Subtotal Construction Cost	$ 400,000
Mobilization (10%)	$ 40,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 200,000
Contingency (30%)	$ 120,000
Property & Easements	$ 160,000
Total Estimated Cost	$ 920,000

5.2.15 Area 15 – Master Planning Alternatives and Recommendation

Area 15 is a branched pipe network. The studied limits start just North of Gray St. and follow the system downstream to its outfall into an open channel just north of Queen Elizabeth Dr. Currently structural flood risks occur for one structure off Gray St. and one structure off W. Johnson St. Roadway flood risks exist for Gray St., W. Johnson St., and N. Harrison Ave. After discussions with the Town's staff it was determined that a recommendation to address all flooding risks for the studied storms was desired. A single alternative was developed for this area.

The alternative for this area calls for the five most upstream pipe segments of the main drainage branch to be replaced with 24-in RCP from its existing 18-in RCP. An illustration of the proposed pipe changes can be seen on the attached map titled "Area 15 – Alternative 1 Upstream Proposed Improvements".

This alternative meets the service goal of maintaining flow below the crown of pipes and addressing all flooding risks in the area. Easement costs are minimized by using the existing alignment. Permitting will require a low level of effort because the improvements are all pipe upgrades. Sewer and water utility lines appear to be less of an issue in this area and are expected to take a minor level of effort to deal with. The EPCC of this alternative is $208,000.

Town of Cary
Basinwide Drainage System Analysis and Water Surface Profile Determination

Area 15 Proposed Master Plan Improvements
Alternative 1

Probable Construction Cost Estimate
Dewberry
April 7, 2006

Item No.	Description	Unit	Rate	Quantity	Amount
23	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (6 - 8 FT DEEP)	LF	$ 108	389	$ 42,100
10	24 inch - REINFORCED CONCRETE PIPE, W/TRENCHING & BEDDING (4 - 6 FT DEEP)	LF	$ 98	217	$ 21,400
81	Catch Basins w/Frame and Grate	Each	$ 1,500	6	$ 9,000
138	10 inch Diameter and Smaller Water	Each	$ 5,000	2	$ 10,000
127	Asphalt or Bituminous Streets	SY	$ 40	31	$ 1,200
131	Curb and Gutter Replacement	LF	$ 20	570	$ 11,400

Estimate prepared by: Sheila Reeves, P.E.

Comments:

Note: Utility Relocations (along Harrison Ave). 2 - Water Main crossings shown in GIS

Subtotal Construction Cost	$ 95,000
Mobilization (10%)	$ 10,000
Engineering, Const. Staking, As-Built and Misc. Professional Services (50%)	$ 48,000
Contingency (30%)	$ 29,000
Property & Easements	$ 25,975
Total Estimated Cost	$ 208,000

5.3 Recommendation Summary and Project Prioritization

Analysis of recommended alternatives was performed to decide which projects are likely to provide the best benefit to cost ratio to the Town in comparison to the other recommended projects. Organizing the recommended projects will assist the Town in prioritizing when the projects will be completed based on available funding. The main criteria that were used to develop the prioritization matrix include:

- Number of roads with a level of service being improved to greater than the 10-year future storm event
- Number of structures with a level of service being improved to greater than the 10-year future storm event
- Cost of the Project
- Permitting Difficulty
- Easement Cost
- Level of Utility Interference

Each alternatives base data for the prioritization matrix, as shown in Table 19, was compared against the other alternatives to determine the project that would entail the lowest cost, least utility interference, and minimal permitting difficulty to provide the greatest reduction in structure and roadway flooding. The criteria in determining how each alternative compared to the other is summarized below. (High point values assigned when rating an alternative based on any criteria reduces the alternatives priority.)

1. Cost – An incremental cost range was used to give each alternative a point value with the highest cost receiving the highest number of points. This factor was weighted as a possible 30 out of 100 points.
2. Structure Flood Risk – Point values were associated with each alternative for the number of structures that would experience reduced flood risk from improvements, where a lesser number of structures results in an alternative receiving higher point values. This factor was weighted as a possible 30 out of 100 points.
3. Roadway Flood Risk – Point values were associated with each alternative for the number of roadways that would experience reduced flood risk from improvements, where a lesser number of roadways results in an alternative receiving higher point values. This factor was weighted as a possible 20 out of 100 points.
4. Easements – An incremental cost range was used to give each alternative a point value with the highest cost receiving the highest number of points. This factor was weighted as a possible 5 out of 100 points.
5. Permitting – A three tier system, with point values of 1, 3, or 5, was used to represent a low, moderate, or high level of difficulty. This factor was weighted as a possible 5 out of 100 points.
6. Utilities – A three tier system, with point values of 1, 3, or 5, was used to represent a low, moderate, or high level of utility conflict resolution. This factor was weighted as a possible 10 out of 100 points.

Table 20 summarizes the point values defined for each alternative, and ranks the alternatives based on the recommended prioritization of the alternatives. Alternative 2 for Area 10 was chosen as the highest priority system to be completed. The preliminary planning level cost of $604,000 to improve this area should provide the Town with the fewest permitting, utility, and easement issues, while resulting in a relatively high reduction in the number of structures and roads experiencing flood risks as compared to the Town's desired level of service. A future price index is also provided in Table 21 assuming a 3.5% annual increase the estimated cost of construction. It is intended that the Town use the prioritization

matrix, criterion for selecting the alternatives, and EPCC as a tools to determine the best way to balance the implementation of these projects with available funds and public policy.

Table 19 Criterion Evaluation for Selected Alternatives

Area	Cost ($)	Structure Flood Risk (# Removed)	Roadway Flood Risk (# Removed)	Easements ($)	Permitting (Low, Moderate, High)	Utilities (Low, Moderate, High)
2	$ 65,000	2	1	$ 12,215	Moderate	Low
3	$ 685,000	3	3	$ 96,900	Low	Moderate
4	$ 647,000	7	3	$ 150,800	Moderate	Moderate
5	$ 167,000	0	2	$ 14,900	Low	Low
6	$ 773,000	3	2	$ 20,500	High	high
8	$1,678,000	17	5	$ 258,100	Moderate	Moderate
9	$ 282,000	1	4	$ 8,600	Low	Low
10	$ 604,000	9	4	$ 160,150	Moderate	Low
12	$ 107,000	1	2	$ 12,215	Low	Low
13	$ 867,000	5	2	$ 160,000	Low	Moderate
15	$ 208,000	2	3	$ 25,975	Low	Low

Table 20 Selected Alternative Priority Ranking Matrix

Area	Chosen Alternative	Cost (30 pts)	Structure Flood Risk (30 pts)	Roadway Flood Risk (20 pts)	Easements (5 pts)	Permitting (5 pts)	Utilities (10 pts)	Total (100-pts)	Priority (Highest to Lowest)
10	Alternative 2	11	14	5	4	3	1	38	1
9	Alternative 2	5	28	5	0	1	1	40	2
15	Alternative 1	3	26	10	1	1	1	42	3
8	Alternative 2	30	0	0	5	3	5	43	4
12	Alternative 1	1	28	15	1	1	1	47	5
4	Alternative 1	11	17	10	3	3	5	49	6
5	Alternative 1	2	30	15	1	1	1	50	7
2	Alternative 1	0	26	20	1	3	1	51	8
3	Alternative 2	12	24	10	2	1	5	54	9
13	Alternative 1	15	21	15	4	1	5	61	10
6	Alternative 1	14	24	15	1	5	10	69	11

Table 21 Indexed Costs for Each Selected Alternative

	SELECTED ALTERNATIVE						
	Chosen Alternative	Present Cost	Cost Indexing				
		2006	2007	2008	2009	2010	2011
Area 2	Alternative 1	$ 65,000	$ 67,275	$ 69,630	$ 72,067	$ 74,589	$ 77,200
Area 3	Alternative 2	$ 685,000	$ 708,975	$ 733,789	$ 759,472	$ 786,053	$ 813,565
Area 4	Alternative 1	$ 647,000	$ 669,645	$ 693,083	$ 717,340	$ 742,447	$ 768,433
Area 5	Alternative 1	$ 167,000	$ 172,845	$ 178,895	$ 185,156	$ 191,636	$ 198,344
Area 6	Alternative 1	$ 773,000	$ 800,055	$ 828,057	$ 857,039	$ 887,035	$ 918,082
Area 8	Alternative 2	$ 1,678,000	$ 1,736,730	$ 1,797,516	$ 1,860,429	$ 1,925,544	$ 1,992,938
Area 9	Alternative 2	$ 282,000	$ 291,870	$ 302,085	$ 312,658	$ 323,601	$ 334,928
Area 10	Alternative 2	$ 604,000	$ 625,140	$ 647,020	$ 669,666	$ 693,104	$ 717,363
Area 12	Alternative 1	$ 107,000	$ 110,745	$ 114,621	$ 118,633	$ 122,785	$ 127,082
Area 13	Alternative 1	$ 867,000	$ 897,345	$ 928,752	$ 961,258	$ 994,902	$ 1,029,724
Area 15	Alternative 1	$ 208,000	$ 215,280	$ 222,815	$ 230,613	$ 238,685	$ 247,039

6.0 Discussion of Results and Recommended Future Activities

6.1 Extended Discussion of Results

The Town initiated the project to determine flood risk and potential improvements in an area with known flooding problems. The study indicated a significant number of structures and roads at risk to the flooding in both the large open channel reaches and the open and piped sections of the stormwater inventory. It is likely other sections of Town have similar conditions and potential flood risks. However, none of the studied areas are included on FEMA mapping, which is the primary regulatory tool for flooding risk.

The value of property within the Town is relatively high and addition of structures that are at flood risk represents a potential substantial financial risk. All of the identified flooding risks in this study are located outside of FEMA mapped floodplains, but extensions of FEMA for only three (3) miles of stream identified 100-year water surface elevations on 281 parcels, 75 of which are not yet developed. This study and similar studies could be used to generate 100-year water surface elevations to be locally regulated or incorporated into the FEMA mapping in other known problem areas or Town wide. This may require ordinances changes or submittal to FEMA or the NFMP. It should be noted that annual flood insurance policy payments that would be associated with residential development on the undeveloped parcels is less than cost to conduct the open channel portion of this study.

A significant percentage of the flooding risk in this study approximately 80 structures is located along stormwater pipe systems which will not benefit from inclusion of studies in the NFIP, so only local regulation changes can prevent similar conditions. Section 6.2 recommends requirements as part of developments permits to determine downstream conveyance capacity. The TCAP may have benefited from this analysis and this type of analysis may prevent costly future capital improvement project to alleviate problem areas in other areas not yet developed.

6.2 Recommended Future Activities

In addition to implementation of the recommend capital improvement projects outlined in Table 13 of Section 5.3, the result of the study indicates a number of potential future activities.

The recommended alternatives represent a large capital improvement effort and the Town should explore implementation of these alternatives in combination with other on-going or planned projects. Combination of the projects could minimize permitting effort and timelines, reducing mobilization costs associated with construction activities and will help reduce the number of separate instances of impacts to citizens through traffic changes, utility interruption and general inconveniences such as noise and aesthetics.

To aid in development and implementation of the selected alternatives, Town staff should coordinate planning meetings, with Town staff and Town Council to develop an implementation schedule within the Town's five-year Capital Plan. This will facilitate the start of the capital improvement process, which is consistent with Town's goal and the one of the goals of the results of this study, which is reduction of flooding risk within the TCAP.

The highest priority project currently has greatest overall potential cost to benefit ratio, as define in Section 5 herein, and current conditions include 14 structures and four (4) roadways in flooding risk. The

Town should initiate preliminary design for this project to accelerate the implementation schedule, so delays between funding and construction can be minimized by removing the engineering and/or permitting phase of work.

The Town should initiate several outreach initiatives based on the results herein. The flooding risk and modeling results can be disseminated in a variety of ways, and two (2) are recommended. The Town should hold a Public Information Hearing to discuss the results of the study with TCAP residents and property owner, or alternatively, the Town could develop and distribute a Public Information Pamphlet to the TCAP property owners. Additionally, the Town is recommended to coordinate with individual property owners regarding specific flooding risk and proposed improvements, as appropriate, for each parcel.

It is recommended the Town determine which projects will require full funding from the Town and projects requiring public matching funds. Once, these systems are identified, the prioritization matrix should be augmented to account for efforts associated with public input and coordination and the reduction in Town of Cary cost based on matching amounts. Also, as matching funds will require additional coordination and an unknown funding timeline, early identification of these systems is essential to eventual development of the capital improvement.

The Town should inventory existing permit data for buildings identified to be located within the newly determined 100-year floodplain. For structures without FFEs, the Town should initiate survey to acquire the FFEs. Based on the results of the inventory and FFE survey, the property owners should notified of the risk associated with the 100-year storm event. If a significant number of structures are at risk for flooding damage, the Town should consider include the 100-year floodplains and data in the NFIP so requirements for flood insurance for building with mortgages implemented. Otherwise, the Town should encourage the property to acquire flood insurance based on the new data and FFE information.

The Town should review the recommended alternatives and identify any projects that may require downstream improvements. If any projects are identified, the Town should initiate study to identify solutions and related cost of these improvements.

The Town has a number of developed areas and developing areas where flooding is a concern. The Town should conduct additional master plan studies to identify problem areas and potential improvements. Since a number of problems identified in this study are related to undersized pipes, the Town should consider requiring development permit request to conduct similar studies to determine capacity of downstream infrastructure to convey the existing discharge and increased discharges resulting from the proposed development.

Appendix A – Location Maps

Appendix B - Hydrology Basin Maps

Appendix C - Hydrology Summary Data

Appendix C - 1 – Land Use Summaries by Basin

BLACK CREEK WATERSHED	Existing		Future		
	Area	Percent of Total Area	Area	Percent of Total Area	Difference in Total Percentage (+/-)
Commercial	5	1	21	4	**3**
High Density Residential	25	5	7	1	**3**
Low Density Residential	205	37	140	25	**12**
Medium Density Residential	5	1	159	29	**28**
Single Family/Multi-Family/Inst	0	0	5	1	1
Industrial Medium	25	5	35	6	**2**
Office/Institutional	51	9	47	9	1
Open Space, Good Condition	16	3	21	4	1
Road	96	18	89	16	**1**
Utility	0	0	0	0	0
Vacant	82	15	0	0	**15**
Very Low Density Residential	19	3	0	0	**3**
Mixed Non-Residential	0	0	4	1	1
Multi-Family	0	0	5	1	1
Commercial - Light	0	0	5	1	
Mixed Residential Non-Resident	0	0	3	1	1
Institutional	0	0	12	2	**2**
Water	0	0	0	0	0
Mixed Single-multi family residence	0	0	0	0	0
Agriculture/Forest	19	3	0	0	**3**

COLES BRANCH WATERSHED	Existing		Future		
	Area	Percent of Total Area	Area	Percent of Total Area	Difference in Total Percentage (+/-)
Commercial	11	3	13	3	1
High Density Residential	9	2	13	3	1
Low Density Residential	151	36	129	30	**5**
Medium Density Residential	22	5	42	10	**5**
Single Family/Multi-Family/Inst	0	0	2	0	0
Industrial Medium	27	6	24	6	1
Office/Institutional	14	3	36	8	**5**
Open Space, Good Condition	16	4	4	1	**3**
Road	89	21	90	21	0
Utility	0	0	0	0	0
Vacant	58	14	0	0	**14**
Very Low Density Residential	3	1	0	0	1
Mixed Non-Residential	0	0	1	0	0
Multi-Family	0	0	33	8	**8**
Mixed Residential Non-Resident	0	0	17	4	**4**
Institutional	0	0	0	0	0
Water	0	0	0	0	0
Mixed Single-multi family residence	0	0	11	2	**2**
Agriculture/Forest	21	5	10	2	**3**

SWIFT CREEK WATERSHED	Existing		Future		
	Area	Percent of Total Area	Area	Percent of Total Area	Difference in Total Percentage (+/-)
Commercial	29	4	23	4	1
High Density Residential	58	9	67	10	**1**
Low Density Residential	265	40	256	39	**2**
Medium Density Residential	19	3	41	6	**3**
Single Family/Multi-Family/Inst	0	0	21	3	**3**
Industrial Medium	20	3	15	2	1
Office/Institutional	45	7	16	2	**4**
Open Space, Good Condition	26	4	15	2	**2**
Road	99	15	99	15	0
Utility	0	0	0	0	0
Vacant	70	11	0	0	**11**
Very Low Density Residential	26	4	0	0	**4**
Mixed Non-Residential	0	0	17	3	**3**
Multi-Family	0	0	2	0	0
Mixed Residential Non-Resident	0	0	20	3	**3**
Institutional	0	0	41	6	**6**
Water	0	0	0	0	0
Mixed Single-multi family residence	0	0	29	4	**4**

WALNUT CREEK WATERSHED	Existing		Future		
	Area	Percent of Total Area	Area	Percent of Total Area	Difference in Total Percentage (+/-)
Commercial	36	6	15	3	**4**
High Density Residential	14	2	2	0	**2**
Low Density Residential	225	40	240	42	**2**
Medium Density Residential	67	12	61	11	**1**
Single Family/Multi-Family/Inst	0	0	36	6	**6**
Industrial Medium	21	4	12	2	**2**
Office/Institutional	29	5	7	1	**4**
Open Space, Good Condition	26	5	14	2	**2**
Road	98	17	96	17	1
Utility	0	0	0	0	0
Vacant	38	7	0	0	**7**
Very Low Density Residential	7	1	0	0	**1**
Mixed Non-Residential	0	0	3	0	0
Multi-Family	0	0	8	1	**1**
Mixed Residential Non-Resident	0	0	40	7	**7**
Institutional	0	0	3	1	1
Water	0	0	4	1	1
Mixed Single-multi family residence	0	0	24	4	**4**

Appendix C - 2 – Existing CN Values and Impervious Area Summary

LU_CODE	SOIL_TYPE	AMCII_CN	AVG_IMP	DESC
AGR/FOR	A	30.0		Agriculture/Forest
AGR/FOR	B	55.0		Agriculture/Forest
AGR/FOR	C	70.0		Agriculture/Forest
AGR/FOR	D	77.0		Agriculture/Forest
AGR/FOR	U	55.0		Agriculture/Forest
AGR/FOR	W	98.0		Agriculture/Forest
COM	A	89.0	85.0	Commercial
COM	B	92.0	85.0	Commercial
COM	C	94.0	85.0	Commercial
COM	D	95.0	85.0	Commercial
COM	U	92.0	85.0	Commercial
COM	W	98.0	85.0	Commercial
HDR	A	77.0	65.0	High Density Residential
HDR	B	85.0	65.0	High Density Residential
HDR	C	90.0	65.0	High Density Residential
HDR	D	92.0	65.0	High Density Residential
HDR	U	85.0	65.0	High Density Residential
HDR	W	98.0	65.0	High Density Residential
LAK	A	98.0		Water
LAK	B	98.0		Water
LAK	C	98.0		Water
LAK	D	98.0		Water
LAK	U	98.0		Water
LAK	W	98.0		Water
LDR	A	51.0		Low Density Residential
LDR	B	68.0		Low Density Residential
LDR	C	79.0		Low Density Residential
LDR	D	84.0		Low Density Residential
LDR	U	68.0		Low Density Residential
LDR	W	98.0		Low Density Residential
MDR	A	57.0		Medium Density Residential
MDR	B	72.0		Medium Density Residential
MDR	C	81.0		Medium Density Residential
MDR	D	86.0		Medium Density Residential
MDR	U	72.0		Medium Density Residential
MDR	W	98.0		Medium Density Residential
MXD	A	83.0		Single Family/Multi-Family/Inst
MXD	B	89.0		Single Family/Multi-Family/Inst
MXD	C	92.0		Single Family/Multi-Family/Inst
MXD	D	94.0		Single Family/Multi-Family/Inst
MXD	U	89.0		Single Family/Multi-Family/Inst
MXD	W	98.0		Single Family/Multi-Family/Inst
OFC/IND	A	81.0	66.0	Industrial-Medium
OFC/IND	B	88.0	66.0	Industrial-Medium
OFC/IND	C	92.0	66.0	Industrial-Medium

LU_CODE	SOIL_TYPE	AMCII_CN	AVG_IMP	DESC
OFC/IND	D	93.0	66.0	Industrial-Medium
OFC/IND	U	88.0	66.0	Industrial-Medium
OFC/IND	W	98.0	66.0	Industrial-Medium
OFC/INS	A	63.0	40.0	Office/Institutional
OFC/INS	B	76.0	40.0	Office/Institutional
OFC/INS	C	84.0	40.0	Office/Institutional
OFC/INS	D	87.0	40.0	Office/Institutional
OFC/INS	U	76.0	40.0	Office/Institutional
OFC/INS	W	98.0	40.0	Office/Institutional
PKS	A	39.0		"Open Space, Good Condition"
PKS	B	61.0		"Open Space, Good Condition"
PKS	C	74.0		"Open Space, Good Condition"
PKS	D	80.0		"Open Space, Good Condition"
PKS	U	61.0		"Open Space, Good Condition"
PKS	W	98.0		"Open Space, Good Condition"
UTL	A	78.0	60.0	"Industrial, Light"
UTL	B	86.0	60.0	"Industrial, Light"
UTL	C	90.0	60.0	"Industrial, Light"
UTL	D	92.0	60.0	"Industrial, Light"
UTL	U	86.0	60.0	"Industrial, Light"
UTL	W	98.0	60.0	"Industrial, Light"
VAC	A	36.0		Vacant
VAC	B	60.0		Vacant
VAC	C	73.0		Vacant
VAC	D	79.0		Vacant
VAC	U	60.0		Vacant
VAC	W	98.0		Vacant
VLDR	A	44.0		Very Low Density Residential
VLDR	B	65.0		Very Low Density Residential
VLDR	C	77.0		Very Low Density Residential
VLDR	D	82.0		Very Low Density Residential
VLDR	U	65.0		Very Low Density Residential
VLDR	W	98.0		Very Low Density Residential
ROAD	A	86.0	80.0	Transportation including ROW
ROAD	B	91.0	80.0	Transportation including ROW
ROAD	C	93.0	80.0	Transportation including ROW
ROAD	D	94.0	80.0	Transportation including ROW
ROAD	U	98.0	80.0	Transportation including ROW
ROAD	W	91.0	80.0	Transportation including ROW

Appendix C - 3 – Future CN Values and Impervious Area Summary

LU_CODE	SOIL_TYPE	AMCII_CN	AVG_IMP	DESC
AGR/FOR	A	30.0		Agriculture/Forest
AGR/FOR	B	55.0		Agriculture/Forest
AGR/FOR	C	70.0		Agriculture/Forest
AGR/FOR	D	77.0		Agriculture/Forest
AGR/FOR	U	55.0		Agriculture/Forest
AGR/FOR	W	98.0		Agriculture/Forest
COM	A	89.0	85.0	Commercial
COM	B	92.0	85.0	Commercial
COM	C	94.0	85.0	Commercial
COM	D	95.0	85.0	Commercial
COM	U	92.0	85.0	Commercial
COM	W	98.0	85.0	Commercial
HDR	A	77.0	65.0	High Density Residential
HDR	B	85.0	65.0	High Density Residential
HDR	C	90.0	65.0	High Density Residential
HDR	D	92.0	65.0	High Density Residential
HDR	U	85.0	65.0	High Density Residential
HDR	W	98.0	65.0	High Density Residential
LAK	A	98.0		Water
LAK	B	98.0		Water
LAK	C	98.0		Water
LAK	D	98.0		Water
LAK	U	98.0		Water
LAK	W	98.0		Water
LDR	A	51.0		Low Density Residential
LDR	B	68.0		Low Density Residential
LDR	C	79.0		Low Density Residential
LDR	D	84.0		Low Density Residential
LDR	U	68.0		Low Density Residential
LDR	W	98.0		Low Density Residential
MDR	A	57.0		Medium Density Residential
MDR	B	72.0		Medium Density Residential
MDR	C	81.0		Medium Density Residential
MDR	D	86.0		Medium Density Residential
MDR	U	72.0		Medium Density Residential
MDR	W	98.0		Medium Density Residential
MXD	A	83.0		Single Family/Multi-Family/Inst
MXD	B	89.0		Single Family/Multi-Family/Inst
MXD	C	92.0		Single Family/Multi-Family/Inst
MXD	D	94.0		Single Family/Multi-Family/Inst
MXD	U	89.0		Single Family/Multi-Family/Inst
MXD	W	98.0		Single Family/Multi-Family/Inst
OFC/IND	A	81.0	66.0	Industrial-Medium
OFC/IND	B	88.0	66.0	Industrial-Medium
OFC/IND	C	92.0	66.0	Industrial-Medium

LU_CODE	SOIL_TYPE	AMCII_CN	AVG_IMP	DESC
OFC/IND	D	93.0	66.0	Industrial-Medium
OFC/IND	U	88.0	66.0	Industrial-Medium
OFC/IND	W	98.0	66.0	Industrial-Medium
OFC/INS	A	63.0	40.0	Office/Institutional
OFC/INS	B	76.0	40.0	Office/Institutional
OFC/INS	C	84.0	40.0	Office/Institutional
OFC/INS	D	87.0	40.0	Office/Institutional
OFC/INS	U	76.0	40.0	Office/Institutional
OFC/INS	W	98.0	40.0	Office/Institutional
PKS	A	39.0		"Open Space, Good Condition"
PKS	B	61.0		"Open Space, Good Condition"
PKS	C	74.0		"Open Space, Good Condition"
PKS	D	80.0		"Open Space, Good Condition"
PKS	U	61.0		"Open Space, Good Condition"
PKS	W	98.0		"Open Space, Good Condition"
VAC	A	36.0		Vacant
VAC	B	60.0		Vacant
VAC	C	73.0		Vacant
VAC	D	79.0		Vacant
VAC	U	60.0		Vacant
VAC	W	98.0		Vacant
VLDR	A	44.0		Very Low Density Residential
VLDR	B	65.0		Very Low Density Residential
VLDR	C	77.0		Very Low Density Residential
VLDR	D	82.0		Very Low Density Residential
VLDR	U	65.0		Very Low Density Residential
VLDR	W	98.0		Very Low Density Residential
ROAD	A	86.0	80.0	Transportation including ROW
ROAD	B	91.0	80.0	Transportation including ROW
ROAD	C	93.0	80.0	Transportation including ROW
ROAD	D	94.0	80.0	Transportation including ROW
ROAD	U	98.0	80.0	Transportation including ROW
ROAD	W	91.0	80.0	Transportation including ROW
CB&R	A	74.0		Mixed Non-Residential
CB&R	B	83.0		Mixed Non-Residential
CB&R	C	88.0		Mixed Non-Residential
CB&R	D	91.0		Mixed Non-Residential
CB&R	U	83.0		Mixed Non-Residential
CB&R	W	98.0		Mixed Non-Residential
CLI	A	83.0		Commercial - light
CLI	B	89.0		Commercial - light
CLI	C	92.0		Commercial - light
CLI	D	94.0		Commercial - light
CLI	U	89.0		Commercial - light
CLI	W	98.0		Commercial - light
HDRG	A	75.0	65.0	Multi-Family
HDRG	B	83.0	65.0	Multi-Family

LU_CODE	SOIL_TYPE	AMCII_CN	AVG_IMP	DESC
HDRG	C	88.0	65.0	Multi-Family
HDRG	D	90.0	65.0	Multi-Family
HDRG	U	83.0	65.0	Multi-Family
HDRG	W	98.0	65.0	Multi-Family
HDRM	A	77.0	65.0	Multi-Family
HDRM	B	85.0	65.0	Multi-Family
HDRM	C	90.0	65.0	Multi-Family
HDRM	D	92.0	65.0	Multi-Family
HDRM	U	85.0	65.0	Multi-Family
HDRM	W	98.0	65.0	Multi-Family
HMXD	A	85.0	75.0	Mixed Residential-Non-Resident
HMXD	B	90.0	75.0	Mixed Residential-Non-Resident
HMXD	C	92.0	75.0	Mixed Residential-Non-Resident
HMXD	D	94.0	75.0	Mixed Residential-Non-Resident
HMXD	U	90.0	75.0	Mixed Residential-Non-Resident
HMXD	W	98.0	75.0	Mixed Residential-Non-Resident
INS	A	69.0	50.0	Institutional
INS	B	80.0	50.0	Institutional
INS	C	86.0	50.0	Institutional
INS	D	89.0	50.0	Institutional
INS	U	80.0	50.0	Institutional
INS	W	98.0	50.0	Institutional
MXDR	A	77.0		Mixed single-multi family res
MXDR	B	85.0		Mixed single-multi family res
MXDR	C	90.0		Mixed single-multi family res
MXDR	D	92.0		Mixed single-multi family res
MXDR	U	85.0		Mixed single-multi family res
MXDR	W	98.0		Mixed single-multi family res

Appendix C - 4 – Existing and Future Curve Numbers by Basin

	Subbasin	Existing CN	Future CN
Black Creek	BC1	77.3	78.7
	BC11	73.2	76.9
	BC4	73.1	76.9
	BC3	71.0	73.0
	BC2	66.6	79.5
	BC6	76.3	76.6
	BC8	72.0	74.9
	BC7	74.2	76.1
	BC5	76.8	76.7
	BC10	82.1	85.6
	BC15	76.0	76.5
	BC9	79.2	87.2
	BC13	81.6	84.4
Coles Branch	CB1	71.4	82.2
	CB2	75.3	79.7
	CB5	75.1	75.2
	CB6	72.8	77.3
	CB9	65.6	79.1
	CB7	77.0	79.2
	CB8	73.3	81.0
	CB4	79.2	80.0
	CB10	78.6	85.9
	CB11	78.3	84.4
	CB12	76.4	76.6
	CB13	82.5	81.6

	Subbasin	Existing CN	Future CN
Swift Creek	SW_16	78.7	84.4
	SW_2	75.7	81.9
	SW_1	71.1	84.8
	SW_3	72.4	75.4
	SW_4	73.4	76.7
	SW_5	76.5	82.6
	SW_6	81.5	84.4
	SW_17	72.4	79.0
	SW_7	72.5	74.0
	SW_9	74.1	78.8
	SW_11	74.1	74.8
	SW_21	79.4	81.4
	SW_13	73.6	74.9
	SW_15	74.2	74.2
Walnut Creek	WC1	86.5	88.2
	WC2	80.6	88.3
	WC6	79.7	85.0
	WC3	74.6	78.7
	WC10	78.8	79.6
	WC4	73.2	74.2
	WC7	73.0	73.0
	WC5	77.3	78.2
	WC8	73.8	80.0
	WC9	75.4	75.6
	WC11	72.2	74.5
	WC12	72.3	73.2

Appendix C - 5 – Time of Concentration Calculation Summary

Black Creek

Basin	Acres	Area (ft^2)	T_C (Hrs)	T_C (Min)	T_{Lag} (Hrs)	T_{Lag} (Min)
BC1	0.35	15,181	0.00	0.00	0.00	0.00
BC11	28	1,219,680	0.35	20.77	0.21	12.46
BC4	21	914,760	0.47	28.27	0.28	16.96
BC3	43	1,873,080	0.54	32.47	0.32	19.48
BC2	69	3,005,640	0.54	32.49	0.32	19.49
BC6	40	1,742,400	0.61	36.73	0.37	22.04
BC8	67	2,918,520	1.23	73.58	0.74	44.15
BC7	42	1,829,520	0.97	57.99	0.58	34.79
BC5	64	2,787,840	0.88	53.01	0.53	31.81
BC10	47	2,047,320	0.53	31.91	0.32	19.15
BC15	56	2,439,360	0.48	29.02	0.29	17.41
BC9	32	1,393,920	0.65	38.89	0.39	23.33
BC13	43	1,873,080	0.48	29.06	0.29	17.44

Coles Branch

Basin	Acres	Area (ft^2)	T_C (Hrs)	T_C (Min)	T_{Lag} (Hrs)	T_{Lag} (Min)
CB10	64	2,787,840	0.48	28.71	0.29	17.23
CB11	49	2,134,440	0.70	42.04	0.42	25.22
CB6	49	2,134,440	0.49	29.18	0.29	17.51
CB7	25	1,089,000	0.33	19.96	0.20	11.98
CB13	18	784,080	0.54	32.41	0.32	19.44
CB12	47	2,047,320	0.48	28.81	0.29	17.29
CB4	18	784,080	0.30	18.18	0.18	10.91
CB8	29	1,263,240	0.53	31.98	0.32	19.19
CB9	11	479,160	0.49	29.20	0.29	17.52

Swift Creek

Basin	Acres	Area (ft^2)	T_C (Hrs)	T_C (Min)	T_{Lag} (Hrs)	T_{Lag} (Min)
SW_16	40	1,742,400	0.44	26.46	0.26	15.88
SW_2	48	2,090,880	0.34	20.67	0.21	12.40
SW_1	44	1,916,640	0.58	34.98	0.35	20.99
SW_3	35	1,524,600	0.40	24.02	0.24	14.41
SW_4	55	2,395,800	0.33	20.04	0.20	12.02
SW_5	29	1,263,240	0.27	16.26	0.16	9.76
SW_6	50	2,178,000	0.39	23.67	0.24	14.20
SW_17	42	1,829,520	0.38	22.74	0.23	13.64
SW_7	42	1,829,520	0.23	13.77	0.14	8.26
SW_9	55	2,395,800	0.47	27.98	0.28	16.79
SW_11	43	1,873,080	0.39	23.52	0.24	14.11
SW_21	63	2,744,280	0.61	36.43	0.36	21.86
SW_13	55	2,395,800	0.30	18.06	0.18	10.84
SW_15	62	2,700,720	0.47	27.99	0.28	16.79

Walnut Creek

Basin	Acres	Area (ft^2)	T_C (Hrs)	T_C (Min)	T_{Lag} (Hrs)	T_{Lag} (Min)
WC5	48	2,090,880	0.51	30.33	0.30	18.20
WC11	57	2,482,920	0.60	36.23	0.36	21.74
WC9	76	3,310,560	0.82	49.28	0.49	29.57
WC8	45	1,960,200	0.63	37.52	0.38	22.51
WC7	24	1,045,440	0.45	27.05	0.27	16.23
WC4	31	1,350,360	0.55	33.05	0.33	19.83
WC10	47	2,047,320	0.55	33.15	0.33	19.89
WC3	46	2,003,760	0.45	26.88	0.27	16.13

Appendix C - 6 – Time of Concentration Flow Paths

Appendix C - 7 – Stage Storage Routing Information for WC5 Attentuation

Pond Dimensions

Initial Elevation	432	ft

Stage	Storage
ft	acres
432	0
434	0.12
436	0.6
438	1.81
440	3.46
442	5.27
444	7.29
446	9.09
448	11.26
450	13.05
454	15

Outlet Orifice

Center Elevation	433.8	ft
Cross Sectional Area	120	ft^2
Discharge Coefficient	0.6	

Spillway

Crest Elevation	448	ft
Crest Width	275	ft
Discharge Coefficient	3	

Appendix C - 8 – 50-Acre Calculated Flows

BLACK CREEK PEAK FLOW SUMMARY 1 – OUTLET JBC1												
Basin_ID	**002yrEX**	**010yrEX**	**025yrEX**	**050yrEX**	**100yrEX**	**002yrFU**	**010yrFU**	**025yrFU**	**050yrFU**	**100yrFU**	**10yr_1hrEX**	**10yr_1hr_FU**
BC1	1	2	2	3	3	1	2	2	3	3	2	2
BC2	46	118	166	204	244	100	191	246	290	333	31	91
BC4	24	51	68	81	95	29	58	76	90	103	19	25
BC5	58	118	154	183	212	58	117	154	183	211	45	44
BC6	45	92	121	144	167	46	93	122	145	168	38	39
BC15	73	149	196	233	270	75	152	199	236	273	62	64
TOTALS												
Average	41	88	118	141	165	52	102	133	158	182	33	44
Minimum	1	2	2	3	3	1	2	2	3	3	2	2
Maximum	73	149	196	233	270	100	191	246	290	333	62	91

BLACK CREEK PEAK FLOW SUMMARY 2 – OUTLET JBC3												
Basin_ID	**002yrEX**	**010yrEX**	**025yrEX**	**050yrEX**	**100yrEX**	**002yrFU**	**010yrFU**	**025yrFU**	**050yrFU**	**100yrFU**	**10yr_1hrEX**	**10yr_1hr_FU**
BC3	39	89	121	147	172	44	96	129	155	181	29	35
BC7	31	66	88	105	123	34	71	93	111	129	21	24
BC8	36	81	109	132	155	43	90	120	143	167	17	21
BC9	40	78	100	118	136	56	96	119	137	155	36	60
BC10	76	140	178	208	237	88	154	192	221	251	74	92
BC11	38	82	109	130	152	47	93	121	143	165	31	42
BC13	74	136	174	203	232	83	147	184	214	242	71	86
TOTALS												
Average	48	96	126	149	172	56	107	137	161	184	40	51
Minimum	31	66	88	105	123	34	71	93	111	129	17	21
Maximum	76	140	178	208	237	88	154	192	221	251	74	92

COLES BRANCH PEAK FLOW SUMMARY												
Basin_ID	**002yrEX**	**010yrEX**	**025yrEX**	**050yrEX**	**100yrEX**	**002yrFU**	**010yrFU**	**025yrFU**	**050yrFU**	**100yrFU**	**10yr 1hr EX**	**10yr 1hr FU**
CB1	33	75	101	122	144	58	107	136	159	181	25	55
CB10	96	186	240	283	326	131	226	282	325	368	86	140
CB11	57	111	144	170	196	74	132	166	193	219	49	74
CB12	62	126	165	196	227	63	127	166	197	228	53	54
CB13	30	55	70	82	93	29	54	69	80	92	29	28
CB2	52	108	143	170	198	65	125	161	189	217	43	59
CB4	35	66	85	100	115	36	68	87	102	116	34	36
CB5	36	73	96	114	133	36	73	96	115	133	31	31
CB6	53	116	155	186	218	68	135	176	209	241	42	59
CB7	43	85	111	131	151	48	91	117	138	158	39	46
CB8	31	67	89	107	125	46	86	109	128	146	24	43
CB9	7	20	28	35	41	17	33	43	50	57	5	16
TOTALS												
Average	45	91	119	141	164	56	105	134	157	180	38	53
Minimum	7	20	28	35	41	17	33	43	50	57	5	16
Maximum	96	186	240	283	326	131	226	282	325	368	86	140

SWIFT CREEK PEAK FLOW SUMMARY												
Basin_ID	**002yrEX**	**010yrEX**	**025yrEX**	**050yrEX**	**100yrEX**	**002yrFU**	**010yrFU**	**025yrFU**	**050yrFU**	**100yrFU**	**10yr_1hr_EX**	**10yr_1hr_FU**
SW_1	38	86	117	142	167	96	168	210	243	275	29	103
SW_11	58	122	162	193	225	60	125	165	197	229	47	50
SW_13	82	173	230	275	320	88	181	238	284	329	68	77
SW_15	75	159	211	252	294	75	159	211	252	294	61	61
SW_16	64	123	159	187	215	81	144	180	209	237	58	85
SW_17	53	114	153	184	215	74	142	183	216	248	41	70
SW_2	75	152	199	237	275	100	183	232	271	309	65	103
SW_21	84	161	207	244	280	92	171	218	254	291	75	86
SW_3	42	91	122	147	172	49	101	133	158	184	33	42
SW_4	77	164	218	262	305	92	183	239	283	327	63	83
SW_5	54	107	140	166	191	71	128	161	187	213	50	80
SW_6	95	175	223	260	297	107	189	237	274	311	94	114
SW_7	66	142	190	227	265	72	150	198	236	274	57	65
SW_9	66	139	185	221	257	84	162	210	247	284	53	76
TOTALS												
Average	66	136	180	214	248	82	156	201	237	272	57	78
Minimum	38	86	117	142	167	49	101	133	158	184	29	42
Maximum	95	175	230	275	320	107	189	239	284	329	94	114

WALNUT CREEK PEAK FLOW SUMMARY												
Basin_ID	**002yrEX**	**010yrEX**	**025yrEX**	**050yrEX**	**100yrEX**	**002yrFU**	**010yrFU**	**025yrFU**	**050yrFU**	**100yrFU**	**10yr 1hr EX**	**10yr 1hr FU**
WC1	79	135	168	193	218	84	140	173	198	223	84	93
WC10	65	126	163	192	221	68	129	167	196	225	58	61
WC11	51	114	153	185	217	59	124	164	197	229	40	48
WC12	67	146	195	235	275	70	150	201	241	281	52	56
WC2	78	146	187	219	250	106	177	218	249	281	76	126
WC3	58	121	160	191	222	71	138	178	210	241	47	65
WC4	32	69	93	111	130	34	72	95	114	133	25	27
WC5	65	129	169	200	230	68	133	173	204	234	56	60
WC6	75	143	185	217	249	94	166	208	240	272	68	96
WC7	28	60	80	96	113	28	60	80	96	113	22	22
WC8	43	93	124	149	173	60	114	147	172	197	35	54
WC9	66	137	181	216	251	67	138	182	217	252	51	52
TOTALS												
Average	59	118	155	184	212	67	128	166	195	223	51	63
Minimum	28	60	80	96	113	28	60	80	96	113	22	22
Maximum	79	146	195	235	275	106	177	218	249	281	84	126

Appendix C - 9 – 10-Acre Calculated Flows

AREA 1

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
DP76416031	WC1_1; WC1_2 [A]	39.1	66.8	41.6	69.3	41.6	46.0
9999999992	WC1_146	19.8	33.8	21.0	35.0	21.0	23.3
9999999962	WC1_145_A	7.0	11.9	7.4	12.4	7.4	8.2
DP76420013	WC1_145_B	6.4	10.9	6.8	11.3	6.8	7.5
DP76420010	WC1_145_C	2.6	4.5	2.8	4.7	2.8	3.1
9999999409	WC1_145_D	4.2	7.1	4.4	7.4	4.4	4.9

[A] – WC_1 flows in to WC1_2 through a piped network. Both basins drain to DP76416031

AREA 2

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
DP76416023	WC10_3_A	3.9	7.6	4.1	7.8	3.5	3.7
9999999968	WC10_3_B + WC10_3_C [B]	11.7	22.6	12.2	23.1	10.4	10.9
DP76420063	WC10_4_A	0.1	0.1	0.1	0.1	0.1	0.1
9999999966	WC10_4_B + WC10_4_C [C]	1.5	3.0	1.6	3.0	1.4	1.4

[B] - WC10_3_B and WC10_3_C drain to the same location without being able to be collected in an inlet system
[C] – WC10_4_B and WC10_4_C drain to the same location without being able to be collected in an inlet system

AREA 3

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
9999999599	WC2_8 + WC2_7_A	28.0	52.3	38.0	63.4	27.2	45.2
9999999893	WC2_7_B	3.2	5.9	4.3	7.2	3.1	5.1
DP76419013	WC2_150_A + WC2_149	18.1	33.9	24.6	41.1	17.6	29.2
DP76419002	WC2_150_B	14.5	27.1	19.7	32.9	14.1	23.4
DP76420026	WC2_150_C	5.5	10.2	7.4	12.4	5.3	8.8
DP76420027	WC2_5_A	4.6	8.6	6.3	10.5	4.5	7.4
DP76420028	WC2_5_B	4.0	7.4	5.4	9.0	3.9	6.4
9999999969	WC2_5_C	0.3	0.5	0.3	0.6	0.2	0.4

AREA 4

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
DP76418007	WC6_40_A	1.8	3.4	2.3	4.0	1.6	2.3
DP76418005	WC6_47	0.3	0.7	0.4	0.8	0.3	0.4
DP76418004	WC6_49	0.1	0.2	0.1	0.2	0.1	0.1
DP76419047	WC6_40_B + WC6_50	0.8	1.6	1.0	1.8	0.7	1.1
DP76419049	WC6_54	1.3	2.5	1.7	2.9	1.2	1.7
DP76419051	WC6_95	1.2	2.4	1.6	2.7	1.1	1.6
DP76419053	WC6_57	0.1	0.2	0.2	0.3	0.1	0.2
DP76419054	WC6_97	0.4	0.8	0.5	0.9	0.4	0.5
9999999984	WC6_55	2.0	3.8	2.5	4.4	1.8	2.5
DP76419060	WC6_109	3.0	5.7	3.7	6.6	2.7	3.8
DP76419061	WC6_59	0.7	1.3	0.9	1.5	0.6	0.9
DP76419069	WC6_63 + WC6_61 + WC6_60	1.0	2.0	1.3	2.3	0.9	1.3
DP76419070	WC6_65	0.2	0.4	0.2	0.4	0.2	0.2
DP76419071	WC6_90	0.2	0.4	0.3	0.5	0.2	0.3
DP76418018	WC6_62 + WC6_76	4.7	8.9	5.8	10.3	4.2	6.0
DP76419105	WC6_64	2.8	5.4	3.5	6.2	2.6	3.6
DP76419106	WC6_75	0.4	0.7	0.5	0.8	0.3	0.5
DP76419103	WC6_66	1.6	3.1	2.0	3.6	1.5	2.1
DP76419102	WC6_77	0.7	1.3	0.8	1.5	0.6	0.9
DP76419101	WC6_78	0.7	1.3	0.8	1.5	0.6	0.9
DP76419100	WC6_79	0.3	0.5	0.3	0.6	0.2	0.3
DP76419099	WC6_80	0.3	0.6	0.4	0.7	0.3	0.4
9999999985	WC6_81	1.6	3.1	2.1	3.6	1.5	2.1
DP76419074	WC6_92	4.5	8.6	5.7	10.0	4.1	5.8
DP76419080	WC6_102 + WC6_100 + WC6_98	4.7	8.9	5.9	10.3	4.2	6.0
DP76419085	WC6_85	0.4	0.8	0.6	1.0	0.4	0.6
DP76419090	WC6_107	0.4	0.7	0.5	0.9	0.4	0.5
9999999983	WC6_114_A	2.2	4.2	2.8	4.9	2.0	2.8
9999999544	WC6_114_B	4.1	7.7	5.1	9.0	3.7	5.2
9999999543	WC6_114_C	2.1	4.0	2.7	4.7	1.9	2.7

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
9999999981	WC6_106	7.0	13.3	8.7	15.4	6.3	8.9
9999999537	WC6_114_D	11.3	21.5	14.1	24.9	10.2	14.4
DP76419141	WC6_115	0.4	0.8	0.5	0.9	0.4	0.5
DP76419148	WC6_116	2.0	3.8	2.5	4.4	1.8	2.5
DP76419153	WC6_129	3.5	6.6	4.3	7.6	3.1	4.4
9999999531	WC6_131_B WC6_133_B	2.2	4.2	2.8	4.9	2.0	2.8

AREA 5

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
9999999889	WC8_160_A WC8_161_A	13.5	29.2	18.8	35.7	11.0	16.9
DP76419233	WC8_160_B	2.8	6.0	3.8	7.3	2.2	3.5
DP76419235	WC8_161_B	3.1	6.7	4.3	8.2	2.5	3.9
9999999699	WC8_21_A WC8_21_B	8.0	17.4	11.2	21.3	6.5	10.1

AREA 6

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
9999999884	SW17_9 SW17_6	25.2	54.3	35.2	67.6	19.5	33.3
DP76306159	SW17_28 SW17_29_D	16.0	34.5	22.4	43.0	12.4	21.2
DP76306154	SW17_29_C SW17_29_B	11.7	25.2	16.4	31.4	9.1	15.5
DP76306145	SW4_17_29_A	2.0	4.3	2.4	4.8	1.7	2.2

AREA 7

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
DP76418150	SW4_40 SW4_89 SW4_136 SW4_135 SW4_42 SW4_84 SW4_85	24.8	52.9	29.7	59.0	20.3	26.8
DP76418149	SW4_86	0.4	0.9	0.5	1.0	0.3	0.5
DP76418145	SW4_87 SW4_83 SW4_79	2.2	4.8	2.7	5.3	1.8	2.4
DP76418143	SW4_88	0.2	0.5	0.3	0.6	0.2	0.3
DP76306132	SW4_92 SW4_125 SW4_91 SW4_90 SW4_122 SW4_124	5.6	12.0	6.7	13.4	4.6	6.1
DP76306131	SW4_123	0.4	0.9	0.5	1.0	0.3	0.5
Dp76106130	SW4_80	1.0	2.1	1.2	2.3	0.8	1.0
DP76306127	SW4_127	1.1	2.3	1.3	2.6	0.9	1.2
DP76306125	SW4_128	0.4	0.8	0.4	0.9	0.3	0.4

AREA 8

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
DP76418126	SW2_162	7.9	16.0	10.5	19.3	6.9	10.9
DP76418131	SW2_163_B	9.4	19.0	12.5	22.9	8.1	12.9
DP76418129	SW2_163_C	2.2	4.5	3.0	5.4	1.9	3.0
9999999961	SW2_163_A	6.6	13.3	8.8	16.0	5.7	9.0
DP76418109	SW2_107	1.4	2.9	1.9	3.5	1.3	2.0
DP76418108	SW2_131	2.2	4.5	3.0	5.5	1.9	3.1
DP76418107	SW2_41	0.5	1.1	0.7	1.3	0.5	0.8
DP76418106	SW2_129	1.6	3.2	2.1	3.9	1.4	2.2
DP76418105	SW2_130	0.4	0.8	0.5	0.9	0.3	0.5
DP76418102	SW2_133, SW2_132 SW4_42	3.5	7.2	4.6	8.6	3.0	4.6
DP76418099	SW2_45, SW2_44 SW2_43	3.0	6.1	4.0	7.4	2.6	4.1
DP76418093	SW2_95, SW2_94 SW2_46, SW2_47 SW2_93, SW2_100 SW2_99	1.8	3.6	2.3	4.3	1.5	2.4
DP76418082	SW2_50 SW2_51	1.9	3.9	2.5	4.7	1.7	2.6
DP76418085	SW2_48	1.2	2.5	1.6	3.0	1.1	1.7
DP76418084	SW2_49, SW2_101 SW2_137, SW2_139 SW2_138, SW2_102 SW2_103	9.8	19.9	13.1	23.9	8.5	13.5
DP76418079	SW2_52	1.0	2.1	1.4	2.5	0.9	1.4
DP76418073	SW2_55, SW2_62 SW2_58	0.5	1.0	0.7	1.3	0.4	0.7
DP76418077	SW2_53	1.0	2.1	1.4	2.5	0.9	1.4
DP76418076	SW2_54, SW2_61	2.1	4.3	2.8	5.2	1.8	2.9
9999999959	SW2_140	3.3	5.3	3.5	6.3	2.3	3.6
9999999549	SW2_98, SW2_106 SW2_105, SW2_63 SW2_56, SW2_57 SW2_104, SW2_60 SW2_64, SW2_59	5.3	10.8	7.1	13.0	4.6	7.3
DP76418059	SW2_70	2.6	5.3	3.5	6.3	2.3	3.6

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
DP76418055	SW2_67, SW2_66 SW2_68, SW2_69	2.3	4.8	3.1	5.7	2.0	3.2
9999999548	SW2_109	2.7	5.5	3.6	6.6	2.4	3.7
DP76306090	SW2_108, SW4_110	1.3	2.7	1.7	3.1	1.1	1.6

AREA 9

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
9999999570	SW1_143	4.4	10.0	11.1	19.5	3.4	11.9
9999999579	SW1_144	10.5	23.8	26.5	46.4	8.0	28.5
DP76417012	SW1_39	1.7	3.8	4.2	7.4	1.3	4.5
9999999576	SW1_166 SW1_167_A	7.8	17.6	19.7	34.4	5.9	21.1
9999999573	SW1_167_B	12.3	27.7	31.0	54.2	9.4	33.2
DP76417023	SW1_96	1.4	3.1	3.5	6.1	1.1	3.8
9999999936	SW16_97	19.8	38.0	25.0	44.5	17.9	26.3
DP76418025	SW16_2_D	6.6	12.6	8.3	14.8	6.0	8.7
DP76418021	SW16_2_A	1.1	2.2	1.4	2.5	1.0	1.5
DP76417076	SW16_2_B	6.5	12.5	8.2	14.6	5.9	8.6
DP76417070	SW16_2_C	5.0	9.6	6.3	11.3	4.5	6.6
DP76417068	SW16_1_C	2.9	5.5	3.6	6.5	2.6	3.8
DP76417066	SW16_1_E	7.2	13.8	9.1	16.2	6.5	9.5
DP76417067	SW16_1_D	4.0	7.8	5.1	9.1	3.7	5.4
DP76417057	SW16_1_B	5.4	10.4	6.8	12.2	4.9	7.2
DP76417056	SW16_1_A	5.5	10.6	6.9	12.4	5.0	7.3

AREA 10

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
9999999507	CB11_19_B	9.9	19.3	12.8	22.9	8.5	12.8
9999999511	CB11_19_A	1.8	3.4	2.3	4.1	1.5	2.3
9999999512	CB11_40	3.6	6.9	4.6	8.3	3.1	4.6
9999999510	CB11_41	2.8	5.5	3.7	6.5	2.4	3.7
9999999515	CB11_42 CB11_44 CB11_45	2.4	4.6	3.1	5.5	2.0	3.1
DP76418223	CB11_43 CB11_46 CB11_48 CB11_51 CB11_50	3.1	6.1	4.0	7.2	2.7	4.0
9999999948	CB11_47_A	3.2	6.1	4.1	7.3	2.7	4.1
DP76418218	CB11_47_B	1.7	3.4	2.2	4.0	1.5	2.2
9999999949	CB11_47_C	2.6	5.2	3.4	6.1	2.3	3.4

AREA 11

Node Name	Contributing Basin	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
DP76418209	CB10_16	17.4	33.8	23.8	41.0	15.6	25.4
DP76418277	CB10_22	4.1	8.0	5.6	9.7	3.7	6.0
DP76418263	CB10_24_A	4.2	8.2	5.8	10.0	3.8	6.2
DP76418207	CB10_23	7.3	14.2	10.0	17.3	6.6	10.7
9999999530	CB10_24_D	5.1	10.0	7.0	12.1	4.6	7.5
9999999528	CB10_24_B	4.9	9.5	6.7	11.5	4.4	7.1
9999999527	CB10_24_C	12.0	23.2	16.4	28.2	10.7	17.5

AREA 12

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
DP76414092	BC5_44 BC5_61	13.2	26.8	13.2	26.6	10.2	10.0
9999999585	BC5_60	2.5	5.0	2.5	5.0	1.9	1.9
DP76414087	BC5_59	0.8	1.5	0.8	1.5	0.6	0.6
DP76414084	BC5_58	0.3	0.6	0.3	0.6	0.2	0.2
DP76414081	BC5_57 BC5_56	3.1	6.3	3.1	6.3	2.4	2.4
9999999592	BC5_55_B	1.9	3.8	1.9	3.7	1.4	1.4
9999999945	BC5_55_A	1.8	3.8	1.8	3.7	1.4	1.4
DP76414043	BC5_54	1.0	1.9	1.0	1.9	0.7	0.7
9999999591	BC5_53	1.3	2.7	1.3	2.7	1.0	1.0
9999999587	BC5_40	7.7	15.7	7.7	15.6	6.0	5.9
9999999589	BC5_42	9.1	18.5	9.1	18.4	7.1	6.9
9999999590	BC5_23_B	1.1	2.3	1.1	2.3	0.9	0.8
9999999586	BC5_23_A	5.5	11.2	5.5	11.1	4.3	4.2

AREA 13

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
DP76415201	BC13_49_A	12.5	23.1	14.1	24.9	12.0	14.6
9999999377	BC13_49_B	6.2	11.4	6.9	12.3	5.9	7.2
9999999986	BC13_51_A	13.1	24.1	14.7	26.0	12.6	15.2
9999999406	BC13_51_B	9.2	16.9	10.3	18.2	8.8	10.7
DP76415129	BC13_52 BC13_50	28.7	52.7	32.2	57.0	27.5	33.4
9999999405	BC13_47	4.3	7.9	4.8	8.5	4.1	5.0
DP76415133	BC9_48_A	1.7	3.3	2.4	4.1	1.5	2.5
9999999404	BC9_48_C BC9_46_A	9.2	17.9	12.9	22.0	8.3	13.8
DP76415114	BC9_48_B	10.3	20.1	14.4	24.7	9.3	15.4
DP76415108	BC9_46_B	10.7	20.8	14.9	25.6	9.6	16.0
9999999402	BC9_46_C	2.8	5.4	3.9	6.7	2.5	4.2
DP76415110	BC9_48_D	5.4	10.5	7.5	12.9	4.8	8.1

AREA 14

Node Name	Contributing Basin	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
9999999500	BC10_37	19.6	36.1	22.7	39.7	19.1	23.7
9999999496	BC10_45	19.1	35.3	22.2	38.8	18.6	23.2
9999999493	BC10_43_C	2.7	4.9	3.1	5.4	2.6	3.2
9999999987	BC10_43_B	9.7	17.8	11.2	19.6	9.4	11.7
DP76415100	BC10_43_A	15.7	28.9	18.2	31.8	15.3	19.0
9999999490	BC10_38	9.2	17.0	10.7	18.7	9.0	11.2
9999999988	BC Basins 9 & 13	114.0	214.0	139.0	243.0	107.0	146.0
9999999494	BC7_31	2.4	5.1	2.6	5.5	1.6	1.9

AREA 15

Node Name	Contributing Basins	Q002yrEX	Q010yrEX	Q002yrFU	Q010yrFU	10yr_1hrEX	10yr_1hr_FU
DP76414191	BC15_62_A BC15_63	11.5	23.6	11.9	24.0	9.8	10.1
DP76414192	BC15_62_B	0.5	1.1	0.5	1.1	0.4	0.5
DP76414176	BC15_64	0.8	1.6	0.8	1.6	0.7	0.7
DP76414194	BC15_65	1.0	2.0	1.0	2.1	0.9	0.9
DP76414175	BC15_66	0.7	1.4	0.7	1.4	0.6	0.6
DP76414174	BC15_67_A	0.3	0.7	0.4	0.7	0.3	0.3
DP76414173	BC15_67_B BC15_69 BC15_68	1.2	2.5	1.3	2.6	1.1	1.1
DP76414172	BC15_70_A	1.1	2.2	1.1	2.2	0.9	0.9
DP76414171	BC15_73_A BC15_71 BC15_70_B	2.0	4.0	2.0	4.1	1.7	1.7
DP76414116	BC15_73_B BC15_75 BC15_74 BC15_72 BC15_39 BC15_70_C BC15_35	33.9	69.2	34.8	70.6	28.8	29.7

BLACK CREEK PEAK FLOW COMPARISON SUMMARY

Basinwide Drainage System Analysis and Water Surface Profile Determination

Town of Cary

December 2005

Peak Flows (cfs) at Watershed Outlet 1(JBC1)			
	Q_100EX	%Difference	Notes
Dewberry	918	na	12 subbasins (avg area = 47ac)
Rural Regression	532	73	
Urban Regression	603	52	
FEMA / State	na	na	

Support Information

Watershed Area =	***250***	acres	0.39	sq mi (from GIS overlay)
Watershed %Impv =	***18***	%		

Peak Flows (cfs) at Watershed Outlet 2 (JBC3)			
	Q_100EX	%Difference	Notes
Dewberry	1010	na	12 subbasins (avg area = 47ac)
Rural Regression	569	77	
Urban Regression	824	23	
FEMA / State	na	na	

Support Information

Watershed Area =	***302***	acres	0.47	sq mi (from GIS overlay)
Watershed %Impv =	***31.5***	%		

COLES BRANCH PEAK FLOW COMPARISON SUMMARY

Basinwide Drainage System Analysis and Water Surface Profile Determination
Town of Cary
December 2005

Peak Flows (cfs) at Watershed Outlet			
	Q_100EX	**%Difference**	**Notes**
Dewberry	1598	na	12 subbasins (avg area = 47ac)
Rural Regression	642	149	
Urban Regression	869	84	
FEMA / State	N\A	N/A	Limited Detail/No information

Support Information

Watershed Area =	***422***	acres	0.66	sq mi
Watershed %Impv =	***21***	%		(from GIS overlay)

SWIFT CREEK PEAK FLOW COMPARISON SUMMARY

Basinwide Drainage System Analysis and Water Surface Profile Determination
Town of Cary
December 2005

Peak Flows (cfs) at Watershed Outlet			
	Q_100EX	**%Difference**	**Notes**
Dewberry	2062	na	14 subbasins (avg area = 47ac)
Rural Regression	754	173	
Urban Regression	1243	66	
FEMA / State	1908	8	Basin SWC7 (DA=1.02) w/ 5-min timestep, CNs reduced by 11%

Support Information

Watershed Area =	***663***	acres	1.04	sq mi
Watershed %Impv =	***27***	%		(from GIS overlay)

WALNUT CREEK PEAK FLOW COMPARISON SUMMARY

Basinwide Drainage System Analysis and Water Surface Profile Determination

Town of Cary

December 2005

Peak Flows (cfs) at Watershed Outlet			
	Q_100EX	**%Difference**	**Notes**
Dewberry	1603	na	12 subbasins (avg area = 47ac)
Rural Regression	713	125	
Urban Regression	1217	32	
FEMA / State	1408	14	Flow ratioed from Basin WC1 (DA=1.12) w/ 5-min timestep, CNs reduced by 11%

Support Information

Watershed Area =	***567***	acres	0.89	sq mi
Watershed %Impv =	***33***	%		(from GIS overlay)

Appendix D - Hydraulic Supplemental Data and Results

Appendix D - 1 – Peak Discharges Used in Open Channel Modeling

Coles Branch

	Peak Discharges (cfs)				
River Station	100-Year	2-Year	10-Year	25-Year	50-Year
18752	326	96	186	240	283
17965	513	150	292	378	446
16733	595	170	335	436	516
16131	789	213	434	571	680
14889	922	249	507	667	795
14232	988	258	536	710	849
13968	1185	310	644	853	1019
12589	1481	387	803	1064	1272
12069	1580	408	853	1133	1355
11701	1598	409	861	1144	1371

Swift Creek Tributary 7

	Peak Discharges (cfs)				
River Station	100-Year	2-Year	10-Year	25-Year	50-Year
24185	762	188	410	546	654
22778	1265	321	685	908	1087
21109	1979	555	1115	1449	1713
19586	2022	540	1115	1465	1744
17974	1968	533	1087	1427	1696
16941	2062	556	1135	1490	1774

Walnut Creek

	Peak Discharges (cfs)				
River Station	100-Year	2-Year	10-Year	25-Year	50-Year
104559	275	67	146	195	235
103538	487	117	257	345	416
102575	660	161	350	469	564
101654	892	223	479	638	766
100828	1191	294	639	851	1020
100742	1396	338	749	997	1195
99644	1538	372	831	1103	1320

Walnut Creek Tributary

	Peak Discharges (cfs)				
River Station	100-Year	2-Year	10-Year	25-Year	50-Year
1714	251	66	137	181	216

Appendix D - 2 – Boundary Conditions for Water Surface Profile Determination

Summary of Boundary Conditions for HEC-RAS Model

Swift Creek Tributary 7

Effective Data (in NGVD 29)

Cross Section	Stationing (ft)	10 yr WSEL	100 yr WSEL	500 yr WSEL	FW WSEL
E	15830	391.1	393.3	397.0	394.1

Used in the Model (in NAVD 88 after a conversion factor of -0.88 is applied)

Cross Section	Stationing (ft)	2 yr WSEL	10 yr WSEL	25 yr WSEL	50 yr WSEL	100 yr WSEL	FW WSEL
E	15830	389.0	390.2	390.6	391.2	392.4	393.2

Coles Branch

Effective Data (in NAVD 88)

Cross Section	Stationing (ft)	10 yr WSEL	50 yr WSEL	100 yr WSEL
10976.1	10976.1	367.35	368.44	368.90

Used in the Model (in NAVD 88)

Cross Section	Stationing (ft)	2 yr WSEL	10 yr WSEL	25 yr WSEL	50 yr WSEL	100 yr WSEL
10976.1	10976.1	366.17	367.35	367.97	368.44	368.90

Walnut Creek

Effective Data (in NAVD 88)

Cross Section	Stationing (ft)	10 yr WSEL	50 yr WSEL	100 yr WSEL
99420.6	99420.6	450.17	451.55	451.95

Used in the Model (in NAVD 88)

Cross Section	Stationing (ft)	2 yr WSEL	10 yr WSEL	25 yr WSEL	50 yr WSEL	100 yr WSEL	FW WSEL
99420.6	99420.6	448.93	450.17	450.69	451.55	451.95	452.95

Appendix D - 3 – Computed Water Surface Elevations from HEC-RAS

Coles Branch

River Station	Water Surface Elevations (ft)				
	100 Year	2 Year	10 Year	25 Year	50 Year
Effective 10976	**368.9**		**367.35**		
10976	368.9	366.68	367.35	367.97	368.44
11701	373.15	371.94	372.76	372.87	372.96
12069	377.06	374.52	375.56	376.3	376.74
12589	381.24	378.66	380.38	380.68	380.97
12805	383.84	381.32	382.88	383.34	383.64
13192	389.08	387.45	388.32	388.6	388.89
13504	392.34	390.24	391.37	391.88	392.09
13797	397.93	394.08	395.75	396.52	397.19
13917	Carrousel Ln.				
13968	405.07	398.87	401.04	402.47	403.65
14232	405.4	400.28	402.12	403.14	404.11
14407	408.95	405.92	407.36	408.04	408.5
14574	Madison Ave.				
14627	415.5	410.71	412.96	414.26	415.15
14889	416.33	414.31	415.63	415.98	416.16
15231	420.46	418.54	419.3	419.79	420.17
15336	Oscar Ln.				
15398	423.91	418.18	420.37	422.88	423.62
15632	425.06	423.02	424.44	424.74	424.87
16131	431.72	429.09	429.97	430.67	431.21
16312	434.49	431.58	433.21	433.86	434.22
16733	438.28	436.51	437.14	437.54	437.91
17125	442.78	440.35	441.67	442.18	442.51
17160	Access Road				
17190	450.47	443.73	446.08	447.56	449.38
17319	450.54	444.52	446.06	447.54	449.45
17556	450.74	447.77	448.52	449.1	449.88
17621	Dixon Ave.				
17652	452	450.64	451.45	451.74	451.94
17849	453.2	450.86	452.25	452.75	453
17922	Dirt Lot				
17965	455.06	454.29	454.55	454.78	454.94
18094	456.65	455.03	455.94	456.25	456.46
18146	Railroad				
18165	471.96	462.27	471.39	471.64	471.8
18389	471.96	462.26	471.39	471.65	471.8
18752	471.93	468.05	471.36	471.61	471.77

Swift Creek Tributary 7

River Station	Water Surface Elevations (ft)				
	100 Year	2 Year	10 Year	25 Year	50 Year
Effective E - 15830	**392.4**		**390.2**		**391.2**
15830	392.4	389	390.2	390.6	391.2
16072	393.14	390.27	391.69	392.21	392.57
16287	394.26	391.47	393	393.57	393.93
16502	395.56	392.55	394.09	394.72	395.16
16673	396.16	393.65	394.79	395.32	395.73
16812	398.37	395.79	397.06	397.66	398.07
16823	399.15	395.71	397.46	398.22	398.67
16878	Maynard Rd.				
16916	405.06	401.27	404.48	404.77	404.87
16935	405.11	401.28	404.49	404.79	404.9
16941	405.12	401.29	404.5	404.8	404.91
16971	405.14	401.3	404.51	404.81	404.93
16980	Footbridge				
16988	405.16	401.31	404.52	404.82	404.95
17118	405.17	401.31	404.52	404.83	404.96
17292	405.21	401.35	404.54	404.85	404.99
17573	405.29	401.46	404.57	404.9	405.05
17802	405.42	401.75	404.64	405	405.17
17974	405.65	403.02	404.81	405.19	405.39
18137	405.87	403.62	404.98	405.38	405.61
18356	406.87	404.59	405.86	406.3	406.6
18541	407.38	405.34	406.4	406.83	407.11
18809	408.48	406.85	407.74	408.03	408.26
19010	409.59	408.11	408.72	409.13	409.37
19177	411.05	409.51	410.46	410.67	410.86
19383	412	410.17	411.18	411.55	411.79
19586	414.5	411.8	413.15	413.74	414.15
19842	415.79	413.34	414.68	415.16	415.5
19991	416.17	413.6	414.8	415.33	415.81
20107	417.97	415.8	417.23	417.54	417.76
20237	419.41	417.86	418.48	418.89	419.17
20308	420.83	417.94	418.17	420.29	420.63
20369	Glendale Ave.				
20413	422.86	421.01	421.99	422.44	422.66
20464	422.87	420.87	421.87	422.4	422.64
20550	423.87	421.37	423.1	423.44	423.65
20589	424.91	422.67	423.91	424.33	424.64
20640	Glendale Ave.				
20677	426.15	424.09	425.16	425.59	425.88
20743	426.29	424.17	425.26	425.71	426.01
20906	426.72	424.47	425.62	426.1	426.42

River Station	Water Surface Elevations (ft)				
	100 Year	2 Year	10 Year	25 Year	50 Year
21109	430.81	427.25	428.95	429.66	430.09
21307	433.11	428.98	430.95	431.87	432.55
21412	433.1	428.89	430.93	431.86	432.55
21590	433.19	430.38	431.18	431.92	432.63
21784	433.81	431.83	432.78	433.12	433.44
21930	434.19	432.52	433.1	433.55	433.87
22165	437.24	435.31	436.52	436.75	436.98
22365	439.3	437.21	438.29	438.77	439.07
22538	440.02	438.31	439.16	439.52	439.78
22612	442.53	440.77	441.62	441.93	442.21
22673	Dixon Ave.				
22778	444.2	443.29	443.87	444.06	444.15
22821	445.21	443.51	444.33	444.68	444.94
22914	445.38	443.59	444.47	444.84	445.11
23007	445.78	443.59	444.74	445.24	445.58
23043	446.9	444.87	445.87	446.43	446.68
23089	Heater Dr.				
23130	451.88	450.57	451.21	451.55	451.68
23182	451.93	450.58	451.23	451.58	451.72
23364	452.01	450.59	451.27	451.64	451.79
23520	453.01	451.39	452.06	452.45	452.74
23626	455.41	452.94	454.61	455.02	455.24
23738	West St.				
23853	459.64	458.63	459.12	459.34	459.49
23922	459.68	458.65	459.14	459.38	459.53
24053	460.55	459.14	459.85	460.19	460.37
24185	463.44	461.97	462.62	462.97	463.21

Walnut Creek

River Station	Water Surface Elevations (ft)				
	100 Year	2 Year	10 Year	25 Year	50 Year
Effective 99421	**451.95**		**450.17**		**451.55**
99421	451.95	448.93	450.17	450.69	451.55
99569	453.23	450.04	451.78	452.36	452.76
99615	Clay St.				
99644	455.66	450.44	454.54	454.86	455.32
100000	456.39	453.68	455.18	455.68	456.02
100230	457.13	454.22	455.75	456.33	456.74
100301	Urban St.				
100333	458.24	455.15	457.21	457.68	457.97
100476	458.72	455.83	457.58	458.07	458.41
100742	460.76	458.51	459.48	460.18	460.49
100800	Webster St.				
100828	461.42	458.74	460.57	460.97	461.2
101096	462.31	460.65	461.37	461.77	462.07
101104	Footbridge				
101116	462.43	460.78	461.53	461.89	462.18
101165	462.65	461.01	461.8	461.93	462.51
101205	E. Park St.				
101240	463.43	461.33	462.84	463.08	463.26
101654	466.4	464.73	465.49	465.93	466.18
101901	467.6	465.44	466.43	466.73	467.33
101980	468.58	466.02	467.48	468.1	468.34
101021	Fairview Rd.				
102073	470.42	469.18	469.86	470.11	470.28
102162	470.85	468.99	469.38	469.8	470.65
102331	472.37	470.16	471.7	472.31	472.35
102575	474.73	472.45	473.57	473.94	474.29
102643	Walnut St.				
102704	477.31	475.09	476.52	476.84	477.09
102959	477.84	475.3	476.85	477.27	477.57
103026	Byrum St.				
103050	478.88	476.08	478.42	478.62	478.76
103538	479.98	478.67	479.2	479.53	479.78
103876	483.51	480.38	480.97	481.15	481.35
103980	Dowell Dr.				
104018	484.65	483.78	484.25	484.37	484.48
104559	487.9	486.71	487.39	487.62	487.76

Walnut Creek Tributary

River Station	Water Surface Elevations (ft)				
	100 Year	2 Year	10 Year	25 Year	50 Year
This model starts using Normal Depth as its Boundary Conditions					
36	465.51	464.17	464.85	465.14	465.34
284	468.39	466.91	467.65	467.96	468.19
323	Warren Avenue				
377	472.59	467.84	471.14	472.31	472.45
468	472.6	468.01	471.16	472.33	472.46
586	473.76	472	472.79	473.52	473.65
627	Maynard Pond Dam				
709	480.35	479.79	480.03	480.16	480.26
1042	480.35	479.79	480.03	480.16	480.26
1327	480.3	479.78	480.01	480.13	480.22
1714	481.22	479.99	480.63	480.89	481.06

Appendix D - 4 – Floodplain Mapping

Appendix E - Hydraulic Grade Line Supporting Data

Appendix E - 1 – Reach Boundary Conditions

Area 1

Storm	WSEL
10yr Future	437.79
10yr Existing	437.68
2yr Future	436.63
2yr Existing	436.38
10yr 1hr Future	436.63
10yr 1hr Existing	436.38

Area 2

Storm	WSEL
10yr Future	448.15
10yr Existing	448.04
2yr Future	446.99
2yr Existing	446.74
10yr 1hr Future	446.99
10yr 1hr Existing	446.74

Area 3

Storm	WSEL
10yr Future	454.37
10yr Existing	454.17
2yr Future	449.49
2yr Existing	448.55
10yr 1hr Future	449.49
10yr 1hr Existing	448.55

Area 4

Storm	WSEL
10yr Future	460.64
10yr Existing	460.57
2yr Future	459.49
2yr Existing	458.74
10yr 1hr Future	459.49
10yr 1hr Existing	458.74

Area 5

Storm	WSEL
10yr Future	472.51
10yr Existing	472.37
2yr Future	471.22
2yr Existing	470.98
10yr 1hr Future	471.22
10yr 1hr Existing	470.98

Area 6

10yr Future	462.82
10yr Existing	462.62
2yr Future	462.15
2yr Existing	461.97
10yr 1hr Future	462.15
10yr 1hr Existing	461.97

Area 7

10yr Future	453.5
10yr Existing	453.3
2yr Future	452.5
2yr Existing	452.1
10yr 1hr Future	452.5
10yr 1hr Existing	452.1

Area 8

10yr Future	443.54
10yr Existing	443.37
2yr Future	443.04
2yr Existing	442.83
10yr 1hr Future	443.04
10yr 1hr Existing	442.83

Area 9

10yr Future	430.1
10yr Existing	429.6
2yr Future	428.4
2yr Existing	427.8
10yr 1hr Future	428.4
10yr 1hr Existing	427.8

Area 10

Storm	WSEL
10yr Future	451.66
10yr Existing	451.45
2yr Future	451.00
2yr Existing	450.64
10yr 1hr Future	451.00
10yr 1hr Existing	450.64

Area 11

Storm	WSEL
10yr Future	471.61
10yr Existing	471.36
2yr Future	468.42
2yr Existing	468.05
10yr 1hr Future	468.42
10yr 1hr Existing	468.05

Area 13*

Storm	WSEL
10yr Future	434
10yr Existing	433.59
2yr Future	432.2
2yr Existing	431.65
10yr 1hr Future	431.5
10yr 1hr Existing	432.35

*based off of the HGL from xsec 9999999988 in Area 14
compared with FlowMaster, all values +/- 0.5 feet using slope of 0.011 and irregular cross section shape

Appendix F - HGL Results, Level of Service Tables, and Area Maps

Appendix F - 1 – Area 1
Appendix F - 2 – Area 2
Appendix F - 3 – Area 3
Appendix F - 4 – Area 4
Appendix F - 5 – Area 5
Appendix F - 6 – Area 6
Appendix F - 7 – Area 7
Appendix F - 8 – Area 8
Appendix F - 9 – Area 9
Appendix F - 10 – Area 10
Appendix F - 11 – Area 11
Appendix F - 12 – Area 12
Appendix F - 13 – Area 13
Appendix F - 14 – Area 14
Appendix F - 15 – Area 15

Appendix G – HGL Profiles

Appendix G - 1 – Area 1
Appendix G - 2 – Area 2
Appendix G - 3 – Area 3
Appendix G - 4 – Area 4
Appendix G - 5 – Area 5
Appendix G - 6 – Area 6
Appendix G - 7 – Area 7
Appendix G - 8 – Area 8
Appendix G - 9 – Area 9
Appendix G - 10 – Area 10
Appendix G - 11 – Area 11
Appendix G - 12 – Area 12
Appendix G - 13 – Area 13
Appendix G - 14 – Area 14
Appendix G - 15 – Area 15