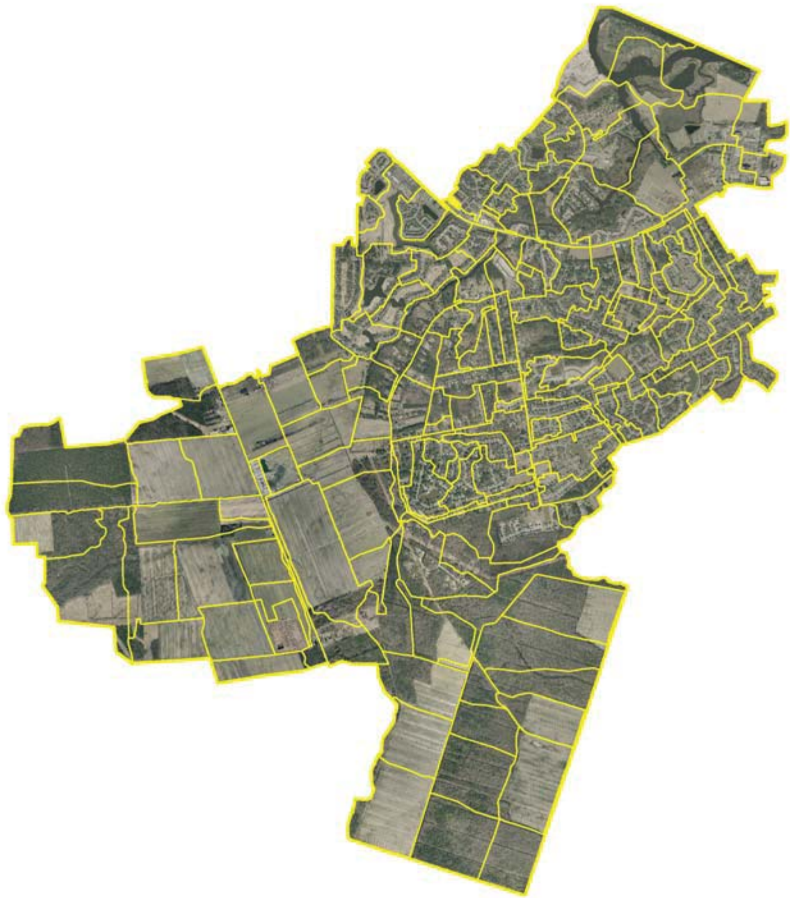
Storm Water Management Model

Bells Mill Creek Watershed MDPU



Master Drainage Plan

April 2009

Revised January 29, 2010

US Army Corps of Engineers
Norfolk District

Chesapeake VIRGINIA
Department of Public Works

URS
URS Corporation

Table of Contents

Figures

Tables

Appendices

Storm Water Management Model

Bells Mill Creek Watershed MDPU

Chesapeake, VA

URS No. 11657375

Executive Summary

Engineers from the U.S. Army Corps of Engineers, City of Chesapeake and URS Corporation have completed a drainage study of the Bells Mill Creek Watershed using the Storm Water Management Model (SWMM) computer program.

The analytical procedure is based on computing localized flood volumes resulting from design rainfall events such as the 2-, 5-, 10-, 25-, 50- and 100-year storms. The watershed is analyzed using modeling configurations to quantify flooding associated with both existing and future watershed conditions. Drainage improvement alternatives are carefully evaluated with respect to their potential impact to the entire watershed. The improvement alternatives are then given further consideration based on construction feasibility and financing constraints, with the focus on the entire watershed rather than on a few individual components. The advantage of this approach is that the entire drainage system can be evaluated on a consistent, system-wide basis.

The process of identifying candidate drainage improvement projects is based on trial-and-error modeling techniques. The watershed is analyzed using anticipated future land use and imperviousness, and locations and volumes of computed flooding are identified in the modeling.

After analyzing existing and potential problems in this watershed, the engineering team has identified nine specific projects that can alleviate future flooding in the subject watershed, and also analyzed a future bridge replacement of the Bells Mill Road over Bells Mill Creek Bridge. Some of these projects are not considered Master Drainage Facilities (MDF's) because their contributing drainage area is not greater than 320 acres. Preliminary cost opinion computations, provided in Appendix E, indicate that the Master Drainage Facilities are financially feasible. These projects can be carried forward as Capital Improvements Projects with some assurance that the impacts on the watershed as a whole have already been adequately considered. Portions of some projects can potentially be constructed as part of private development initiatives with little or no cost to the City. It is also important to keep in mind that some of these improvements may need to be modified, as wetlands regulations, flooding issues, soil properties, economic considerations, and Federal Aviation Administration (FAA) regulations may effectively impact future development.

The City has identified several locations, throughout the subject watershed, in which chronic flooding has been an issue. The SWMM model confirmed flooding at these locations, as indicated in Figure 8. The

City confirmed that other locations have either already received recent drainage improvements, or that previously reported flooding was due to maintenance issues.

There are many combinations of drainage improvements that can be evaluated in any watershed. While a substantial effort has been applied to develop this study, it is by no means exhaustive. The intent of this undertaking was not only to develop sound alternatives for watershed improvements, but also to leave the underlying data files and computer models so that they can be used in a straightforward manner in the future.

The maximum computed water surface elevation at each modeled node, and peak computed discharge at each modeled link are presented in Appendices C and D, respectively, for existing and future conditions.

Portions of this watershed associated with roadway or development projects have been evaluated by the City over the past several years. Some studies have been completed to address specific problems as described elsewhere in this report. The modeling conducted as part of this Master Drainage Plan Update incorporates the previously prescribed improvements where possible, either directly or with modifications.

FEMA flood insurance studies and rate maps are the definitive source of floodplain limits and elevations. The SWMM models developed for this drainage study are specific design scenarios based on 2-, 5-, 10-, 25-, 50-, and 100-year rainfall events—THEY ARE NOT TO BE CONSTRUED AS INDICATIVE OF EXPECTED WATER SURFACE ELEVATIONS FOR THE PURPOSES OF FLOODPLAIN MANAGEMENT AND/OR INSURANCE REQUIREMENTS. The SWMM models developed for this study could be adapted for use in the National Flood Insurance Program and submitted to FEMA for approval, but until they are subjected to that process the published flood insurance studies and rate maps remain fully in effect.

Background

URS was directed by the City of Chesapeake and the U.S. Army Corps of Engineers to conduct a study on the area of Bells Mill Creek Watershed covering approximately 6,350 acres.

The Bells Mill Creek Watershed is located in the center of Chesapeake and is bordered on the east by the Cooper's Ditch and Horse Run Ditch East Watersheds, on the south by the Southern Chesapeake Watershed, on the west by New Mill Creek and Camden Mills Watersheds, and the north by the Oak Grove Watershed and Albermarle and Chesapeake Canal. Runoff from the Bells Mill Creek Watershed discharges into the Southern Branch of the Elizabeth River.

The subject watershed was delineated into 162 subcatchments in order to compute and distribute runoff throughout the entire watershed. The northeast portion of Bells Mill Creek Watershed is well developed with future expectations for development for the southern and western portions. Two-thirds of the southern portion, known as the Bosher Farm Site, is under an EPA (Environmental Protection Agency) consent order to remain undeveloped. The remaining third (having development potential) will be required to provide on-site detention facilities to handle the additional stormwater runoff. The Bells Mill Creek Watershed has recently received several stormwater improvements including the Cedar Road Bridge and concrete ditch outfall, the Herring Ditch (upstream end) Improvements and the Glen Abbey outfall. Other stormwater improvement projects that are scheduled for the near future include the replacement of the Bells Mill Road over Bells Mill Creek Bridge and the widening of Herring Ditch (downstream end). The City will use this study to verify whether or not these improvements, as previously designed, are adequate. In the case of the Bells Mill Road Bridge replacement, this study presents a minimum span configuration that can be used to adequately pass flows from extreme storm events. In the case of Herring Ditch the City would prefer on-site detention, if deemed necessary, in lieu of ditch widening. As a general rule, the City prefers to create on-site detention facilities rather than disturb natural streams, as well as to handle increased runoff from new development. Due to the close proximity to the Chesapeake Regional Airport, new development should adhere to FAA Advisory Circular No. 150/5200-33B. Beginning in 1997 the FAA greatly tightened restrictions for separation areas, as explained elsewhere in this report. The FAA separation areas are mapped in Figure 10.

This study addresses existing drainage and stormwater issues, as well as expected future conditions. The entire SWMM model has 325 nodes and over 340 links, providing sufficient detail and modeling resolution for master drainage planning purposes.

Two drainage studies, the Bells Mill Master Drainage Study (1985) and the Forest Lakes/Whispering Pines/Vintage Estates/Southwood Drainage Study (2000), were previously completed within this watershed, as summarized below.

The Bells Mill Master Drainage Study was conducted in March of 1985 by Gannett Fleming Corddry & Carpenter. The study concentrated on three main study areas, each draining to a separate crossing of Cedar Road: the main channel, also referred as Herring Ditch, drains approximately 3,650 acres to the twin 6-ft. x 6-ft. box culverts under Cedar Road; Tributaries 2 and 3 drain through twin 54-inch diameter culverts at both Waters Road and Cedar Road; and Horse Run Ditch West drains approximately 482 acres to its confluence with the main channel, some 485 feet upstream of the Bells Mill Road Bridge. Recommended improvements for the area were:

1. Approximately 18,400 feet of substantial channel improvements from Cedar Road to a point about 5,600 feet west of Shillelagh Road.
2. Approximately 915 feet of channel realignment on the downstream side of Shillelagh Road.
3. Five (5) 8-ft. x 6-ft. box culverts under Cedar Road (Herring Ditch outfall).
4. Four (4) 6-ft. x 5-ft. box culverts at the relocated crossing under Shillelagh Road.
5. Approximately 7,200 feet of channel improvements for Tributaries 2 and 3.
6. Two (2) additional 54-inch diameter culverts at Cedar Road (Tributaries 2 and 3 outfalls) for a total of four (4) 54-inch diameter culverts.
7. Two (2) 60-inch diameter culverts at Waters Road.
8. One (1) additional 48-inch diameter culvert at the dirt farm road for a total of three (3) 48-inch diameter culverts.

The City's GIS and construction plans show that all the crossing culverts listed above have been implemented with larger sizes than recommended. In addition, improvements to a few channel sections have been made.

The Forest Lakes/Whispering Pines/Vintage Estates/Southwood Drainage Study by the City of Chesapeake, Department of Public Works Engineering Division was conducted in December 1999 and revised in January 2000. The study focused on investigating the conveyance capacity of ditches due to blockages and vegetative growth at the Vintage Estates, Southwood, and Forest Lakes outfalls and due to the combination of relocated ditches constructed with the City's Brine Line and the resurrection of a previously neglected drainage system on the 1,500-acre Bosher tract south of Whispering Pines which resulted in a significant increase in runoff. Flooding at several locations throughout the system led to the following recommendations:

1. Replace the existing 60-inch and dual 24-inch pipe crossing, located just before the ninety degree bend section (about 1,900 feet north of intersection of Herring Ditch), with a 40-foot concrete bridge.
2. The feeder ditch between Whispering Pines and Herring Ditch must be enlarged to a 25-foot bottom width to accommodate additional flows from the Bosher property.

3. Widen the channel from the double 58-in. x 83-in. pipe crossing under Wittington Drive to match the downstream channel width, approximately 200 feet.
4. Widen the ditch to a 14-foot bottom width between Parker Road and west of Greenfield Lane's cul-de-sac.

Improvement No. 4 has been implemented with a 12-foot bottom width and a culvert crossing the newly constructed subdivisions.

In addition to the previous studies, the City of Chesapeake provided URS with several plan sets for projects within the subject watershed, some of which have been approved for construction but have not yet been completed. As directed by the City, URS modeled these as 'existing' conditions. While some of these developments were not expected to be complete by the end of this study, they were considered existing conditions because the approval of the project assures its near-future development.

The City of Chesapeake surveyed selected points in the subject watershed at the request of URS. These selected survey points are presented in Appendix B. The City also provided URS with GIS-related topographic data. URS utilized these four main sources—past studies, plan sets, survey data, and GIS data to extract channel and infrastructure information, such as inverts, pipe type and sizes, and channel characteristics, throughout the subject watershed.

Methodology

The engineering methodology applied in this study is summarized in a separate document, submitted by URS to the City of Chesapeake in April of 2005, entitled *Master Drainage Plan Methodology*. SWMM modeling is typically used for relatively large-scale studies. It is not generally intended to be used as a design tool for individual projects, due to its complexity and data requirements. Its strength lies in the application of very advanced hydrologic and hydraulic routing computational routines, fed with data from a geographic information system (GIS) and from plans for future roadway and parcel development projects.

This Master Drainage Plan Report presents the findings of the application of this methodology to the subject watershed.

Treatment of Nodal Flooding

The issue of how to handle nodal flooding is important when using or interpreting any rainfall-runoff model, including SWMM. Loosely speaking, nodal flooding occurs when a computed water surface elevation exceeds the maximum defined depth at a point in the system (referred to as a "node").

In previous versions of SWMM (Versions 4.x and earlier), the water leaving the node was treated as an "escape" from the system. However, the treatment of nodal flooding was enhanced in SWMM Version 5 by introducing "nodal ponding" and "nodal surcharge" capabilities. The new nodal ponding option allows the modeler to specify a constant "ponding area" over which <u>nodal surcharges are stored as they escape from the node, then released back into the system as water surface elevations recede</u>. This nodal ponding capability can produce more reliable water surface elevation computations due to the re-introduction of nodal flooding volumes and their continued downstream routing through the drainage system.

The option to compute nodal ponding in SWMM necessitates an approach to treat or develop the ponding area for each node, subject to two considerable limitations. First, the ponding area increases with depth, and in fact at some depth the ponded volume will actually combine with other nearby nodes such that deciding which node has what portion of the surface flooding becomes arbitrary at best. Secondly, it is not feasible to spend the time performing elaborate delineations at each node to compute a constant ponding area that is approximate at best, requires judgment regarding how much area to assign to which node, and ultimately varies with depth. In many locations, the situation is further complicated—when stormwater flows up and out of the ground, it runs down a gutter or downhill flow path to some other location.

SWMM is a one-dimensional model—it can only compute flow depth, discharge and related properties along one-dimensional lines through the drainage network. It cannot compute lateral variations in the flow (such as can be accomplished with two-dimensional surface-flow models). Even if it were possible to precisely compute the ponding area at each node, we are still limited by the use of a one-dimensional model. It is difficult to determine a ponding area with accuracy when the computed water surface elevation exceeds the ground elevation. The problem is further complicated by the difficulty in determining the nominal "ground elevation" in a one-dimensional model.

URS has developed an approach to handle nodal flooding using SWMM Version 5, which we are using on many similar studies. The approach used is to divide the total watershed area by the number of modeling nodes to develop an average ponding area, which is then applied to all nodes that are not directly modeled as storage nodes. This approach is simple, but effective, and because the surface flooding is re-introduced into the drainage system as flood levels decrease, it gives a reliable basis upon which to compute water surface elevations in these models.

Vertical Datum

Unless specifically stated otherwise, the **North American Vertical Datum of 1988 (NAVD88)** was used throughout this study.

Modeling Configurations

Three modeling configurations—Existing Hydraulics with Existing Hydrology (Scenario 1), Existing Hydraulics with Future Hydrology (Scenario 2), and Future Hydraulics with Future Hydrology (Scenario 3)—were developed for this study as described below.

Scenario 1 Existing watershed hydrology with the drainage system configured as it existed in 2008. Channels are modeled using their existing (2008) conditions as well. This is the "Scenario 1" model. The City of Chesapeake requested certain plan sets be considered as 'existing' because they have been approved prior to the start of this study. The following is a list of plan sets and studies, provided by the City, that were used in the existing conditions model (the list includes completed past studies, projects that have been constructed, as well as approved projects not yet constructed):

1. North Rollingwood Estates
2. Dove acres
3. 533 Butterfly Drive - Dove Acres
4. North Rollingwood Estates - Phase 1
5. North Rollingwood Estates - Phase 2
6. Frost Estates
7. Watsons Glen
8. Bells Mill Road Subdivision
9. Grant Estate

10. Aston Park
11. Cross Property
12. Whispering Pines - Phase 2
13. Wadell Estates
14. Brandon Woods
15. Shillelagh Estates
16. Moore's Farm Subdivision
17. Cedar Lakes Shopping Center
18. Wood Development Country Club
19. Covenant Fellowship Church
20. Cedarwood Office Building
21. Forest Lakes/ Whispering Pines/ Vintage Estates/ South Wood Drainage Study
22. Bells Mill Master Drainage Study
23. Herring Ditch Outfall Improvements
24. Bells Hollow Condos
25. Cedar Lakes
26. Saddle Ridge South
27. Twelve Oaks
28. Homemont - Boston Ave. Outfall
29. Bosher Site Map
30. Las Gaviotas Drainage Map
31. Cedar Road Improvements
32. Cedarwood Condos
33. Cedarwood South Subdivision
34. Cedarwood Sec III
35. Forest Lakes/Bellwood (compiled plan sheets)
36. Cedar Road Ph 1, 2 3 1994 Road Bond
37. Cedarwood Estates
38. Cedarwood
39. Cheshire Forest (Phase 4)
40. Las Gaviotas
41. Wilson Heights South
42. Glen Abbey

Scenario 2 Future watershed hydrology with the drainage system configured as it existed in 2008. Channels are modeled using their existing (2008) conditions as well. This is the “Scenario 2” model. This scenario will show the flooding effects of the existing drainage system due to future land use development. In other words, if no improvements are made to the current drainage system and the remainder of the watershed is constructed as described by the City’s 2005 Adopted Land Use Plan, these are the locations and volumes of flooding that can be expected.

Scenario 3 Future watershed hydrology with the future drainage system configured as envisioned by the City of Chesapeake and URS. This is the “Scenario 3” model. This scenario incorporates the drainage from Scenario 2 along with any recommendations from the engineering team to help eliminate flooding on a Master Drainage Facility level (i.e. facilities serving 320 or more acres). In addition, this scenario includes future plans previously identified by the City. The following is a list of plan sets, provided by the City, that were added to the future conditions model:

1. Washington Drive
2. Waters Road Ditch

The recommended improvements should largely reduce flooding at key locations, where feasible, for future conditions. These improvements were developed during this study, are highlighted in Figure 10, and specifically include the following projects:

1. Bosher Site Drainage Improvements
2. West Shillelagh Drainage Improvements
3. East Shillelagh Drainage Improvements
4. Herring Ditch Outfall Improvements
5. Copeland Drive Outfall Improvements
6. Foxgate Outfall Improvements
7. Lake Circle Outfall Improvements
8. Dove Acres Outfall Improvements
9. Horse Run Ditch Outfall Improvements
10. Bells Mill Road Bridge Replacement

This scenario depicts future conditions with strategic drainage and stormwater improvements in place. Additional details and descriptions regarding the improvements are presented elsewhere in this report. Cost opinions are presented in Appendix E.

Modeling Results

The City has identified five locations within the Bells Mill Creek Watershed where chronic flooding has been an issue. These areas include (refer to Figure 6 and/or the GIS files for node locations):

1. Unicorn Trail (Nodes 165 & 166)
2. Forest Lake – Back Lake (Nodes 170 & 175)
3. Glen Abbey (Node 545)
4. Copeland Drive (Node 737)
5. Lake Circle (Node 779)

The SWMM model confirms flooding at both Copeland Drive and Lake Circle locations. The Copeland Drive location has low street elevations. The SWMM model, however, does not verify flooding at the other three locations. Upon further discussion with City staff, it was discovered that the Unicorn Trail area is well known for receiving sheet flow runoff from the neighboring Bosher Farm Site. The City is aware that several culvert crossings on the private property are in need of maintenance or repairs. It is these poorly maintained crossings that are causing overtopping of the banks and flooding downstream to Unicorn Trail. Since improvements have been made to the Herring Ditch (upstream end) and the Glen Abbey outfall, the Forest Lake, Back Lake and Glen Abbey pond, respectively, have not reported any further flooding issues.

The maximum computed water surface elevations at each modeled node and computed peak discharge at each modeled link are presented in Appendices C and D, respectively, for existing and future conditions.

Stable SWMM runs were obtained for all modeling scenarios. Continuity errors ranged from low to very low. URS senior engineers used PCSWMM.NET to review dynamic hydraulic grade lines, checking the hydraulic routing for potential stability problems or any type of flow anomaly. During this QA/QC procedure items were found and addressed, so the final modeling results should be reliable.

Boundary conditions (water surface elevations) at the downstream outfall were established by the City of Chesapeake Public Works division. In all cases, for all return periods, the hydraulic boundary condition was modeled as a constant water surface elevation of 3.60 feet (NAVD88) in the Southern Branch of the Elizabeth River. Due to the natural topography and wide floodplain environment of the Bells Mill Creek, the water surface elevations in the upper portions of this watershed are not very sensitive to the downstream boundary water surface elevation used in these models.

The GIS analysis prepared in support of this modeling indicates that the Bells Mill Watershed will increase from **16.55** to **21.87** percent imperviousness in the future, as indicated in Figures 3 and 4. The procedures used to determine this increase are explained in the *Master Drainage Plan Methodology* (April 2005) report submitted previously. This increase in impervious cover produces greater volumes of stormwater runoff, which have been incorporated into the future conditions models.

During the process of determining imperviousness, URS engineers noticed that there are areas in the City's GIS showing lower imperviousness than actually exists. For example the City's GIS Land Use data indicates areas in the northeast of this watershed, near Cedar Road, having low-density residential (15% imperviousness) land use, when they are in fact varying between middle- to high-density residential (25% to 50% imperviousness). URS and City engineers subsequently agreed that those areas needed to be adjusted in Scenarios 1, 2, and 3 in order to obtain accurate results.

Figures 8, 9, and 11 depict street and property flooding volumes for the 10- and 50-year design storm events. The histograms are not drawn to any scale, but they are proportional, and serve to graphically identify where flooding can be expected under each modeling configuration.

The City does not have to 'fix' all of the flooding represented by the histograms in the figures. Areas such as woodlands, deep ravines, large open spaces, ball fields and parks, and along railroad rights of way often do not require improvements unless there is a specific reason to construct them. It is also important to bear in mind that a 50-year design storm is an extreme event, and that neighborhood drainage systems are typically not required to accommodate 50-year storms.

Flooding complaints, particularly those in residential neighborhoods, often result from maintenance problems such as a clogged pipe or debris in a ditch. In considering whether or not drainage improvements might be required to correct an *existing* deficiency, the model results should indicate a flooding problem, and there should be some flooding history to support the need for improvements. If both of these conditions are not met, then the system maintenance should be reviewed or the preliminary computer models should be carefully scrutinized.

It is also important to understand when reviewing these results that there can be low-lying structures in the watershed that have finished floor elevations below the maximum water surface elevations computed in the SWMM models. In order to estimate whether or not a particular structure will be subject to flooding for a given storm condition, maximum hydraulic grade line elevations in the vicinity should be checked against the finished floor elevation.

As with all models of this size and complexity there is a great deal of detailed information required. Because it is not feasible to collect *all* of the required data, in some locations it is necessary to make educated guesses about inverts and pipe and channel dimensions and geometries. Where future designs and studies will be based on these models, engineers are strongly encouraged to field-verify all items that may critically impact their designs.

The maximum computed water surface elevations at each model node are presented in Appendix C for both existing and future condition scenarios. The blue shading in Tables C-1 and C-2 indicates locations where the maximum computed water surface meets or exceeds the ground elevation for that node. Many of these nodal flooding locations are very small quantity or short duration events. In these SWMM 5 models, the volume of water leaving the node during flooding is computed and summarized for continuity purposes (which allows for a reasonable accounting of flood volume at the node) *and the flooded water is re-introduced into the model for subsequent downstream routing,* as explained in the Treatment of Nodal Flooding section above. If flooding occurs at a choke point in the system, downstream (or nearby) nodes may have computed maximum water surface elevations less than what can actually be expected due to the volume of water being 'held' upstream. With the introduction of Nodal Ponding in SWMM 5, this phenomenon is of less concern than it was in older versions of SWMM. Where computed water surface elevations exceed the ground elevation in these models, water surface elevations in the vicinity should be considered 'approximate'. The main purpose of this ponding approach is to account for local flooding volumes and re-introduce stored water back into the drainage system as water surface elevations recede.

The <u>figures</u> that indicate nodal flood volumes in this report have been filtered so that nodal flood volumes less than 10,000 cubic feet are not represented (because less than 10,000 cubic feet of flooding cannot be practically discerned on the ground—it simply appears as heavy runoff or sheet flow in most cases). <u>Tables</u> C-1 and C-2 have not been filtered at all; where nodal flooding is indicated in many cases the duration and quantity of flooding can be very minor.

The PCSWMM.NET modeling platform contains a very helpful dynamic hydraulic grade line tool that allows the user to view animations of the computed water surface elevations. This dynamic hydraulic grade line tool takes input from a digital interface file at *a specified sampling interval*, for example every 3 minutes in these models. The SWMM routing computations are performed at one-second (or so) intervals, and the output file contains summary information based on *every* time step. If the dynamic hydraulic grade line tool is used to view the results the user should bear in mind that it is based on a sample (one out of every 180 seconds), and therefore the 'peak' values listed by the dynamic hydraulic grade line tool are peaks as sampled using a three-minute interval. The SWMM output data on the other hand contains a summary of the *exact* peak values. The SWMM output file summaries were used to prepare Tables C-1, C-2, D-1, and D-2, as well as the flooding figures in this report.

The modeling results presented in this report are based on the assumption that the drainage and stormwater systems will be well maintained. If debris builds up to block drainage structures, or channels are allowed to fill with silt, flooding will likely be more severe than computed and represented in this report. Debris can be a significant problem in natural channel outfall systems, and should be monitored carefully to ensure that these systems function properly.

FEMA flood insurance studies and rate maps are the definitive source of floodplain limits and elevations in all cases. The SWMM models developed for this drainage study are specific design scenarios based on 2-, 5-, 10-, 25-, 50-, and 100-year rainfall events—THEY ARE NOT TO BE CONSTRUED AS INDICATIVE OF EXPECTED WATER SURFACE ELEVATIONS FOR THE PURPOSES OF FLOODPLAIN MANAGEMENT AND/OR INSURANCE REQUIREMENTS. The SWMM models developed for this study could be adapted for use in the National Flood Insurance Program and submitted to FEMA for approval, but until they are subjected to that process, the published flood insurance studies and rate maps remain fully in effect.

Master Drainage Plan Improvements

The Chesapeake Regional Airport is located to the southwest of the Bells Mill Creek Watershed, just outside the watershed boundary. Stormwater management facilities and water amenities such as detention basins, retention basins, or wetlands can contribute to an increase in wildlife-aircraft strikes because they provide an inviting habitat for waterfowl. The FAA published Advisory Circular No. 150/5200-33B, "Hazardous Wildlife Attractants On or Near Airports," to address this issue. Specific separation distances to be maintained between an airport and the wildlife attractant are recommended by the FAA: 5,000 feet for airports serving piston-powered aircraft; 10,000 feet for airports serving turbine-powered aircraft; and 5 statute miles to protect the airport's airspace. These FAA separation areas are mapped in Figure 10. Guidelines are given by the FAA for the construction of new stormwater management facilities within the recommended separation distances. Stormwater management systems should be designed so as not to create above-ground standing water, and to eliminate glide paths for waterfowl. For detention basins, they should be designed for a 48-hour detention period after storms and stay completely dry between storms. Another guideline is using steep-sided, riprap-lined, narrow, linear-shaped water detention basins to control hazardous wildlife. Physical barriers are recommended to reduce aircraft-wildlife interactions. Another guideline is removing all vegetation around detention basins because they can provide food and cover. These recommendations to minimize wildlife-aircraft strikes should be addressed in development plans. Additional recommendations and details can be found in Advisory Circular No. 150/5200-33B online at http://www.faa.gov/airports_airtraffic/airports/resources/advisory_circulars.

The City of Chesapeake utilizes a 320-acre threshold for candidate Master Drainage Facility (MDF) improvements. If a project services less than 320 acres, it will generally not be constructed as part of the City's Master Drainage Plan.

Ten specific projects were conceived and incorporated into the modeling during the course of this study, four of which will not be considered an MDF improvement because their contributing drainage areas are less than 320 acres. These projects are by no means exhaustive, but they seem to provide a reasonable amount of flooding relief at reasonable costs. All of the projects appear to be feasible from a preliminary planning standpoint, but issues such as future wetlands delineations and the ability to successfully acquire rights-of-way or parcels of land may necessitate some modifications as these projects move forward. The ten projects are shown in Figure 10 and are included in the future modeling scenario (Scenario 3). Refer to Figures 7 and 10 of this report to find node and link numbers and to view the locations of improvements that are referenced in the following project summaries. In some cases, due to the tight proximity of nodes and links and the text size of their labels, it may be easier to view these links and nodes using the GIS files provided with this report.

As indicated in Figures 1 and 3, much of this watershed is currently undeveloped. The timing of future development will affect which projects have to be done when. Figure 4 presents potential future imperviousness based on future build out according to the City's comprehensive land use plan. There are other factors to be considered that will affect future imperviousness, such as legal agreements between the EPA, Corps of Engineers, and the Bosher Farm Site owners restricting development to allow only one-third of the Bosher Farm Site to be developed. Such restrictions have not been through the City's zoning approval process, but will affect how development occurs. Likewise, although zoning may allow development, wetlands restrictions may further limit actual future imperviousness. Modeling for this report reflects potential future increases in imperviousness according to the City's comprehensive plan and rights-of-way for future road projects.

1. Bosher Site Drainage Improvements

Future imperviousness will increase if this site is developed. Given that the existing ground cover is extensively agricultural and forested, future development must carefully address stormwater management to prevent downstream erosion and flooding that could result from substantially increased runoff volumes.

Improvements in this area are intended to handle the majority of the increased runoff volume in new serpentine storage channels. The serpentine configuration has been adopted to lessen long glide paths for waterfowl. The exact layout can be determined in the future, but the shapes depicted should work for stormwater management purposes as long as the same storage volume is provided.

Project components involving improvements along outfall channels should be designed using “stream restoration” techniques in order to minimize potential costs and delays stemming from regulatory review processes

Recommended improvements to this area include:

1. Node 103: Construct a serpentine retention channel along the existing ditch alignment with a normal water surface width of 40 ft, length of 5,000 ft, and side slopes of 2H:1V. Depth from the water surface to top of bank should be 3.8 ft, and the total storage volume is 21.0 acre-feet.
2. Node 105: Construct one 30-in RCP connecting the serpentine channel outfall to the existing ditch at node 105. Disconnect the existing ditch from nodes 105 to 107.
3. From Nodes 105 to 115: Construct trapezoid channel with bottom width of 5 ft and side slopes of 2H:1V.
4. From Node 109 to 115: Widen channel bottom to 10 ft with side slopes of 2H:1V.
5. From Node 115 to 118: Construct double 42-in RCPs under the dirt road and connect to existing ditch. Disconnect the existing ditch from nodes 115 to 117.
6. Node 132: Construct a serpentine retention channel along the existing ditch with normal water surface width of 40 ft, length of 6,900 ft, and side slopes of 2H:1V. Depth from the water surface to top of bank should be 5.3 ft, and the total storage volume is 43.0 acre-feet.
7. From Node 118 to132: Connect the existing channel to the new serpentine channel.
8. Node 137: Construct one 36-in RCP connecting the serpentine channel outfall to the existing ditch.
9. Node 141: Construct a serpentine retention channel along the existing ditch alignment with a normal water surface width of 40 ft, length of 2,830 ft., and side slopes of 2H:1V. Depth from the water surface to top of bank should be 5.0 ft, and the total storage volume is 17.00 acre-feet.
10. Node 143: Construct two 48-in RCPs connecting the serpentine channel outfall to the existing ditch.

Because these improvements are on private property and are only required to support future development, costs for the Bosher Site improvements will be borne by the developer.

2. West Shillelagh Drainage Improvements

The purpose of these improvements is to relieve flooding and also to help reduce downstream flooding. The serpentine configuration has been adopted to lessen long glide paths for waterfowl. The exact layout can be determined in the future, but the shapes depicted should work for stormwater management purposes as long as the same storage volume is provided.

Recommended improvements to this area include:

1. Node 207: Construct a serpentine retention channel along the existing ditch alignment from node 219 to node 211 (note that this retention channel is <u>not</u> connected to the existing channel from 217 to 247). Continue this serpentine channel upstream from 207 to nodes 199 and 203 as indicated in Figure 10. This serpentine channel system should have a normal water surface width of 40 ft, total length of 5,900 ft., and side slopes of 2H:1V. Depth from the water surface to top of bank should be 5.0 ft., and the total storage volume is 33.7 acre-feet.

2. Node 211: Construct one 48-in RCP connecting the serpentine channel outfall to the existing ditch.

The estimated cost for the West Shillelagh Drainage Improvements project is **$ 2,219,163** in 2009 dollars, as summarized in Appendix E. This cost would be reduced if constructed as part of adjacent development projects.

3. East Shillelagh Drainage Improvements

This project will restore the existing channel to its design geometry and relive upstream flooding from future development.

Recommended improvements to this area are:
1. From Node 255 to 259: Clean out and widen the channel bottom (where needed) to 20 ft with side slopes of 2H:1V. This project should be approached as 'maintenance' for regulatory purposes.
2. From Node 259 to 265: Widen the channel bottom to 20 ft with side slopes of 2H:1V.

The estimated cost for the East Shillelagh Drainage Improvements project is **$ 294,992** in 2009 dollars, as summarized in Appendix E. This cost would be reduced if constructed as part of adjacent development projects.

4. Herring Ditch Outfall Improvements

This project will also restore the existing channel to its design geometry and relive upstream flooding from future development.

Recommended improvements to this area are:
1. From Node 265 to 271: Widen the channel bottom to 20 ft with side slopes of 2H:1V.
2. From Node 271 to 273: Replace existing two 24-in RCPs and existing 84-in CMP with two 84-in RCPs.

The estimated cost for the Herring Ditch Outfall Improvements project is **$ 524,214** in 2009 dollars, as summarized in Appendix E. This cost would be reduced if constructed as part of adjacent development projects.

5. Copeland Drive Outfall Improvements

The SWMM models show that flooding at Copeland Drive results from flow reversals at node 743, that back up to Bartell Drive. Improvements for this project are based on 50-year storm analyses, to facilitate appropriate system-wide flood relief consistent with Master Drainage Plan objectives for the larger watershed.

Recommended improvements include:
1. Node 743: Restore the existing detention basin by cleaning out sediment and overgrowth, and excavate the bottom down to elevation 4.5 ft, with side slopes equal to 2H:1V. The new bottom elevation will increase the storage volume to 11.0 acre-feet, without requiring additional property.
2. From Node 743 to 744: Construct one 36-in RCP to form the basin outfall with a Tideflex valve installed to prevent backflow. This outfall should connect to the existing paved ditch.

3. Node 715: Lower invert to 12.00 ft.
4. Node 717: Lower invert to 11.80 ft.
5. From Node 715 to 717: Replace the existing pipe with a 48-in RCP.
6. From Node 717 to 723: Replace the existing 30-in RCP with a 48-in RCP.
7. From Node 723 to 727: Replace the existing 42-in RCP with a 60-in RCP.
8. From Node 727 to 744: Replace the existing pipe with two 54-in RCPs and extend these two 54-in RCPs to bypass the detention basin and outfall to the existing downstream paved ditch.

No cost opinion has been prepared because the contributing upstream area for this project is less than 320 acres.

6. Foxgate Outfall Improvements

Improvements for this project are based on 50-year storm analyses, to facilitate appropriate system-wide flood relief consistent with Master Drainage Plan objectives for the larger watershed.

Recommended improvements include:
1. From Nodes 707 to 709: Replace the existing 42-in RCP with a 60-in RCP.
2. From Nodes 709 to 711: Replace the existing pipe with a 60-in RCP.

No cost opinion has been prepared because the contributing upstream area for this project is less than 320 acres.

7. Lake Circle Outfall Improvements

Improvements for this project are based on 50-year storm analyses, to facilitate appropriate system-wide flood relief consistent with Master Drainage Plan objectives for the larger watershed. The existing lake can provide additional valuable flood storage if the water surface can be lowered by six inches. A new outfall control weir can be used for this purpose, and the additional flood storage provided helps to relieve downstream flooding.

Recommended improvements include:
1. Node 779: Construct a new weir at the lake outfall with a crest elevation at 5.4 ft, and a total effective weir length of 30 ft. A new structure could be constructed to provide weir control and accommodate a new outfall pipe (item 2 below).
2. Node 779 to 781: Replace existing 24-in outfall RCP with a 42-in RCP. This pipe should be placed at elevation of 3.1 ft after the weir.

No cost opinion has been prepared because the contributing upstream area for this project is less than 320 acres.

8. Dove Acres Outfall Improvements

Improvements for this project are based on 50-year storm analyses, to facilitate appropriate system-wide flood relief consistent with Master Drainage Plan objectives for the larger watershed.

Recommended improvements include:
1. Node 786: Lower invert to 8.50 ft.

2. Node 787: Lower invert to 8.30 ft.
3. From Node 786 to 787: Replace the existing 15-in RCP with a 30-in RCP.
4. From Node 787 to 790: Replace the existing 15-in RCP with a 30-in RCP.
5. From Node 790 to 796: Add one 30-in RCP parallel to the existing 24-in x 38-in RECP.
6. From Node 796 to 797: Replace the existing pipe with two 36-in RCPs.

No cost opinion has been prepared because the contributing upstream area for this project is less than 320 acres.

9. Horse Run Ditch Outfall Improvements

Improvements in the Horse Run area become more necessary in the future because as upstream development and drainage improvements take place, more runoff volume will be produced and conveyed downstream.

There are prime real estate parcels remaining to be developed in this area (along Cedar Road), and these outfall improvements have been conceived with the intent of minimizing adverse impacts to future development.

There is an existing low-lying parcel at node 819 that has a structure on it that has been subjected to flooding in the past. This parcel was recently purchased by the Girl Scouts and it is anticipated that the structure will be demolished and the site converted to recreational use with no improvements that will be subject to flood damage. For this reason it is not necessary to make stormwater or structural improvements downstream from Cedar Road.

Recommended improvements include:
1. From Node 791 to 793: Clean out the channel.
2. From Node 795 to 798: Clean out the channel.
3. From Node 798 to 799: Widen the channel bottom to 10 ft. with side slopes of 2H:1V.
4. From Node 799 to 801: Replace the existing triple 48-in RCP with double 5-ft x 7-ft box culverts.
5. From Node 801 to 805: Convert the 5-ft paved ditch to a 35-ft earthen channel with side slopes of 2H:1V. This improvement may be used in part to satisfy green space requirements for future site development, if City regulations so allow.
6. From Nodes 805 to 807: Replace the existing 6-ft x 8-ft box culvert with double 5-ft x 7-ft box culverts.
7. From Nodes 807 to 815: Convert the 5-ft paved ditch to a 35-ft earthen channel with side slopes of 2H:1V. This improvement may be used in part to satisfy green space requirements for future site development, if City regulations so allow.

The estimated cost for the Horse Run Ditch Outfall Improvements project is **$ 1,023,601** in 2009 dollars, as summarized in Appendix E. This cost would be reduced if constructed as part of adjacent development projects.

10. Bells Mill Road Bridge Improvement

The City intends to replace the Bells Mill Road over Bells Mill Creek Bridge, from node 827 to 829, and asked URS to suggest a possible new span configuration within the context of this master drainage plan study. URS engineers working on this assignment have extensive experience with the hydraulic design of bridges, scour analyses, and scour countermeasures design, however this master drainage plan study is not a replacement for the hydrologic and hydraulic analyses that are required to adequately analyze scour potential and provide structural recommendations based on computed scour profiles.

With that important caveat in mind, URS generally reviewed the existing bridge with respect to scour potential and hydraulic configuration. No bridge plans, soil borings, stratigraphy data, or scour inspection reports were provided to URS, but City surveyors confirmed the critical elevations and dimensions of the bridge.

The existing structure is a single-span, vertical-abutment design. The approaches are constructed on fill across the natural floodplain, which can partially block flood flows as water builds up on the upstream side of the bridge and exits through the bridge opening. This redirection of flows along the approaches may contribute to local scour at the abutments. However the roadway approaches are at relatively low elevations, so the potential to build up a high head differential across the bridge opening does not exist. According to the City's GIS topographic data, the roadway approaches on both sides of the bridge are lower than the existing deck elevation of 6.08 feet. Spot elevations in the GIS indicate roadway elevations of 5.14 and 5.02 on the northwest and southeast approaches respectively.

The low chord elevation on the existing bridge is 4.08 feet, and the channel invert under the bridge is approximately minus 6.3 feet. Although local abutment scour may be an issue, extreme flood flows are relieved over the roadway approaches before the bridge can be overtopped.

The annual tide elevation used in this master drainage plan modeling is not entirely suitable for scour analysis in that it provides a backwater effect that serves to retard flows and velocities. URS made 100- and 500-year SWMM runs to develop a potential replacement bridge configuration using a free overflow hydraulic boundary condition at the outfall (node 999). The free overflow boundary condition maximizes the computed velocities through the bridge opening. Another set of runs was made using the annual high tide boundary condition, which maximizes the hydraulic head on both sides of the bridge. Both head and velocity are important in the design of scour countermeasures and the design of the bridge substructure. These runs are saved with project modeling data as "Scenario 4."

URS also made multiple runs to conservatively develop design discharges for the analysis, varying soils and other modeling parameters in the watershed model to develop a feel for the sensitivity of these models for bridge scour analysis. The model is not particularly sensitive to any input parameters, in large part due to the size of the upstream contributing watershed and the storage that is available throughout the watershed.

After modeling multiple trials to test the modeling parameters, URS contacted City staff to find out if there was any history of significant head differential across the bridge opening or significant scour history. The bridge deck and roadway approaches are sufficiently low that we would expect overtopping to have occurred more than once in the recent past (e.g. during Hurricane Floyd, and Tropical Storm Ernesto perhaps), but would expect the flood elevations on both sides of Bells Mill Road to be relatively equal. Likewise we would not be surprised to hear that there was some local abutment scour due to the fill that comprises the roadway approaches. City staff confirmed that there has been a minor abutment scour issue at this site, and there has not been a significant head differential across the roadway at the bridge site.

With these observations in mind, a replacement span having the following configuration was analyzed for Scenario 4:

1. From Nodes 827 to 829: a new bridge having a 50-ft span, with a minimum low cord elevation of 4.5 ft, and riprap placed from elevation 0.0 ft up the new abutments to create 2H:1V abutment slopes. This minimum low chord elevation does not include design freeboard, which may be added as appropriate.

The bridge designers can set the deck and final low chord elevations based on the girder and deck configuration selected, which may in part depend upon utility details in the vicinity of the bridge, availability of right-of-way, permitting requirements for impacted wetlands (if any), and topography in the vicinity of the bridge. The FEMA 100-year flood elevation in this area is 7.60 feet (NAVD88). Roadway elevations nearby are just above 5 feet to the east and west of the bridge. For Master Drainage Plan purposes, as long as the low chord elevation is above 4.5 feet, the bridge should adequately pass upstream runoff. Depending upon the final deck configuration selected by the bridge designers, tidal backwaters (i.e. storm surges and extreme tidal events) may inundate the bridge in the future, as the bridge has been inundated in the past.

With a 50-foot span (10 feet longer than the existing span), and a low chord of 4.5 feet (0.42 feet higher than the existing low chord), and sloping abutments protected by riprap, the following hydraulic results were observed in the Scenario 4 models.

Table 1. Bridge Hydraulics for 50-Foot Span Configuration

Scenario 4 Model	Maximum Water Surface Elevation (ft)		Peak Computed Discharge (cfs)			Peak Computed Velocity (fps)		
	Upstream Face	**Downstream Face**	**Approach Channel 18251**	**Through Bridge 18271**	**Exit Channel 18291**	**Approach Channel 18251**	**Through Bridge 18271**	**Exit Channel 18291**
100-yr, Free Overfall	1.06	0.99	1934	1952	1952	5.87	7.14	4.87
500-yr, Free Overfall	1.65	1.59	2233	2274	2273	5.85	7.56	5.09
100-yr, Annual High Tide	3.98	3.97	1522	1534	1534	1.36	3.69	2.04
500-yr, Annual High Tide	4.08	4.07	1764	1772	1772	1.52	4.22	2.35

The above results are not intended as a substitute for a bridge hydraulic design and scour analysis, but suggest the suitability of a 50-foot span for the replacement design.

The current budget estimate for replacement of this bridge is **$ 1,500,000** in 2009 dollars.

Master Drainage Plan Caveats

The goal of this type of study is not to relieve *all* flooding, but rather to identify Master Drainage Facility improvements that can be feasibly constructed. It is also important to consider that neighborhood and commercial parcel drainage and stormwater systems are neither required nor designed to accommodate flooding from extreme events such as the 50-year storm.

One important caveat to keep in mind is that the system as modeled for this study assumes a well-maintained system. Debris, sediment, pipe collapses and other maintenance issues can cause very real flooding that must be addressed. In this respect, this study highlights *capacity* issues rather than *maintenance* issues (which are best resolved from inspection or citizen reports). There is good reason to create the models in this manner. If poor maintenance conditions are modeled, the capacity problems could easily be masked to the extent that public funds could be spent unnecessarily.

These models should also be useful for obtaining starting hydraulic grade line elevations for design purposes on smaller development projects, and for designing stormwater management BMPs on specific sites. URS is providing the models completed for this study to the City in the hope that future engineering efforts will build upon this effort.

Contact Information

Mr. Sam Sawan, PE (757.382.6101) served as the project manager for the City of Chesapeake on this project. Mr. Mark Mansfield, Chief Planning and Policy Branch, and Mr. Walter Trinkala, Engineering Technical Specialist (757.201.7610) represented the Corps of Engineers, Norfolk District. Mr. John Paine, PE, PH, CFM was the project manager for URS. The modeling evaluations were produced by Hai Tran, EIT. The report and QA/QC was provided by Stephanie Hood, PE. Cost opinion and additional production assistance was provided by Sean Bradberry, and Carol Wilkinson (757.873.0559).