Appendix F - Watershed Modeling Process

Hydrologic, water quality, and hydraulic models were developed to simulate the existing and planned future development conditions in the watershed and to evaluate the benefits of proposed Best Management Practices (BMPs) on the watershed runoff and water quality. The County provided guidelines for the modeling process in the document Technical Memorandum No. 3, Stormwater Model and GIS Interface Guidelines, June 2003. The specific goals established by the County for the modeling process were as follows:

- Predict the existing water quality and flow conditions in the watershed.
- Determine the impacts of development projected to occur in the watershed.
- Quantify the benefits provided by various stormwater management measures.
- Identify stream crossing flooding and improvements.
- Justify the overall benefits of watershed management planning alternatives.

Hydrologic, water quality, and hydraulic models were created to predict the existing and future conditions in the watershed and to evaluate the proposed alternatives. SWMM was used to develop the hydrologic and water quality models which calculated the effects and benefits of low impact development (LID) and best management practices (BMPs) on runoff and water quality. HEC-RAS was used to develop the hydraulic model which calculated the in-stream velocities and water surface elevations used to identify and control flooding locations.

Hydrologic Model Development

The goal of the hydrologic and water quality model was to develop runoff and water quality results specific to each subbasin in the watershed. The results of the model would enhance the decision-making process to locate problem areas and to help determine which stormwater controls should be recommended to address specific problems identified by the model.

The first step in the hydrologic modeling process was to divide the 7,067-acre Little Hunting Creek watershed into 37 smaller subbasins with an average size of approximately 200 acres. The subbasins were created automatically using the digital elevation map (DEM) and the mapping program ArcGIS. ArcHydro Tools for ArcGIS was used to create stream centerlines, connect stream segments, create outlets and create subbasins. Line features were created from raster data with ArcGIS. After the initial delineation, the pour points for the subbasins were adjusted in order to provide a more accurate delineation and the subbasin boundaries were manually adjusted by modifying the boundary lines to correspond to drainage structure mapping. This process resulted in subbasin boundaries that correctly corresponded to the stormwater infrastructure data, reach data, and hydraulic crossings. Some subbasins were only used in the hydrologic and water quality models because the subbasins drained directly into the South Little Hunting Creek tidal area or Potomac River.

After the adjustments were made, the subbasin data was submitted to the county for approval. The county determined that one subbasin next to Huntley Meadows Park should not be included in the Little Hunting Creek Watershed modeling because it appeared to drain to an adjacent watershed.

For modeling purposes, a new delineation was also created that excluded the open water of

Little Hunting Creek from the watershed boundaries because the open water area would not be used in the hydrologic calculations. The parcel file was edited with ArcGIS by removing duplicate parcels within the watershed boundary.

Each of the 37 subbasins was further divided into three separate subareas based on the following criteria:

- Runoff from an area was controlled by a stormwater detention facility
- Runoff from an area was controlled by a stormwater detention and water quality facility
- Runoff from an area had no stormwater controls

The subareas were designated as "A", "B" and "C", respectively. A data file with the approximate location of detention and water quality control facilities was provided by the county and locations of the facilities were adjusted to more accurately reflect the controlled properties within the watershed. Using this parcel file, the parcels were intersected with the subbasin boundary. Subareas were also designated as "D" for areas without parcel data, which included roadway right-of-way. The D area was factored into the A, B, and C areas by summing the parcels (A, B, and C areas) and calculating the percent area for only A, B, and C. The D area for each subarea was then multiplied by the fraction of A, B, and C. The fraction of D area of each parcel area was then added into the A, B, and C areas for each subarea.

For calculating runoff within SWMM, four key parameters were calculated: the area, the width (a parameter that affects the peak runoff rate and the shape of the runoff hydrograph), the slope, and the percent impervious of the ground surface. The subbasin width represents the average width of overland flow in a subbasin. The width was calculated by dividing the subbasin area by the overland flow. The overland flow was determined by taking the flow distance and dividing it by the difference between the most hydrologically distant part of the watershed and the downstream point in the watershed. The flow path was visually determined from the contour file and the stormwater drainage system maps. The slope of the subbasin was determined by taking the most upstream and downstream elevations along the flow path of each subbasin and dividing by the length of the flow path for the watershed. The percent imperviousness was divided between the area that drained directly to the storm drain system and the impervious surface that drained across grassed or wooded ground cover. Only the area that was directly connected to the storm drain system was included in the determination of the impervious surface. The following GIS layers were used to determine the percentage of existing impervious surface:

- Roads (paved and non-paved)
- Parking (paved and non-paved)
- Sidewalks
- Buildings (commercial, residential, industrial)

With the exception of the parcels identified as underutilized or vacant, the future impervious surface layer remained relatively unchanged from the existing land use area. For the impervious areas on the underutilized and vacant parcels, the existing percent imperviousness land use was replaced by a percent impervious surface for either the planned land use or the zoning land use, whichever land use represented the greatest potential impervious area.

The pervious area, PERVA, was calculated by subtracting the total impervious area from the total area of each subbasin. The remainder of each subbasin was the pervious area, PERVA. The NDCIA and the directly connected impervious, DCIA, were calculated as described below. The directly connected impervious surface is defined as the impervious surface that directly drains to the storm drain system. An example of the DCIA surface would be a roadway that drains to an inlet that is part of the storm drain system.

The infiltration parameters were determined and used by SWMM to calculate the amount of infiltration during a storm event using the Horton infiltration method. The primary source of data used for the infiltration parameters was the National Resource Conservation Service (NRCS) State Soils Geographic (STATSGO) database and mapping. The Fairfax County GIS soils database and mapping had incomplete coverage for the watershed. Where the county soils data and the NRCS STATSGO data overlapped, the county data replaced the NRCS Soils data. Each soils polygon had an assigned number from 1 through 4, with 1 assigned to Hydrologic Soils Group "A" and 4 assigned to Hydrologic Soils Group "D". The weighted average for each HSG for each subbasin was calculated using the area of the entire subbasin and the area of the soils polygons within the watershed. The HSG was averaged based on the polygon for the watershed. A weighted HSG for the entire subbasin was calculated by multiplying the area of the polygon by the assigned HSG number. The HSG-area value was divided by the entire area of the subbasin to calculate a weighted HSG number.

The soils information was used to determine three infiltration parameters for the Horton method, the max infiltration rate (WLMAX), the minimum infiltration rate (WLMIN), and the decay coefficient for infiltration capacity (DECAY). The parameters WLMAX and WLMIN were interpolated using the weighted HSG for each subbasin. No interpolation was needed to calculate the DECAY parameter, since it was consistent for each HSG. The weighted WLMAX and WLMIN were then adjusted within ArcGIS, to account for the impervious area that is not directly connected to the storm drain system (NDCIA). Each parameter was also adjusted by multiplying by a factor that was calculated using the equation: Factor = PERVA / (PERVA + NDCIA)

Another parameter that was calculated for the hydrologic model was the overland flow roughness coefficient. The overland flow roughness coefficient is used in the runoff calculations for SWMM-RUNOFF and is different for impervious and pervious surfaces. The pervious surface was calculated based on land cover for the overland flow roughness coefficients. For pre-developed conditions, the pervious overland flow roughness coefficient was set to 0.35 for all subbasins, which corresponded to the land use "Open Space" for pervious surfaces. For the impervious surface for the existing development, the roughness coefficient was set to 0.015. For the existing developed conditions within the watershed, a weighted average was calculated for each subbasin for the pervious roughness coefficient.

The final parameter to be calculated for the hydrologic model was the depression storage. The depression storage, which is water that is stored in depressions on the land surface and becomes neither runoff nor infiltration, was established as 0.10 inches for impervious surfaces and 0.20 inches pervious surfaces. The percentage of impervious area with zero depression storage was set to 25%, which simulates immediate runoff.

Water Quality Model Development

The water quality pollutants modeled for future and existing conditions included the following:

- Biochemical Oxygen Demand (BOD)
- Chemical Oxygen Demand (COD)
- Total Suspended Solids (TSS)
- Total Dissolved Solids (TDS)
- Dissolved Phosphorus (DP)
- Total Phosphorus (TP)
- Total Nitrogen (TN)
- Total Kjeldahl Nitrogen (TKN)

Four parameters were used in SWMM to simulate the buildup and wash-off of the pollutants from the land surface. Two parameters were used to predict the buildup of pollutants and two to predict wash-off from the land surface. The parameter values were provided by the county in the document titled Development of SWMM Water Quality Model Inputs for Fairfax County, Virginia, March 2004.

The two parameters used to represent pollutant build up on the land surface include:

- QFACT(1), which represents the maximum pollutant accumulation on the land was obtained from the Draft Study Memorandum Fairfax County SWMM Land Use Pollutant Loading Parameterization and compiled and interpolated from event mean concentration values from an EPA report titled Considerations in the Design of Treatment Best Management Practices (BMPs) to Improve Water Quality.
- QFACT(2), which is an exponential factor that determines the accumulation rate and how quickly the surface pollutant mass recovers after a storm has washed pollutants off the land surface. QFACT(2) was set to a value of 0.1/year.

The washoff parameters for the wet weather events for the water quality module of RUNOFF were RCOEF and WASHPO. The following describes the parameters:

- RCOEF, a washoff coefficient, was set to a standard 4.6 inches^{-1}.
- WASHPO, an exponential rate factor that is applied to the calculated surface runoff rate, was set to a recommended value of 1.0 inches.

Each subarea was divided into the following land use groups in order to assign water quality parameters within SWMM:

- Residential
- Commercial
- Industrial
- Open space

Two land use maps GIS layers developed for the current land use and the planned land use conditions. When estimating the existing and future landuse area for each subarea, the area within the watershed that was not included in the parcel mapping was apportioned to the parcel land use based on percentage of the overall subarea. The area not within the parcels predominantly included roadway right-of-way. The water quality parameter values for the four residential land uses (estate, low density, medium density, and high density residential) were weighted to calculate one value for residential land use and the water quality parameter values for the two commercial land uses (low and high intensity commercial) were weighted to calculate one value for commercial land use.

As discussed previously, in the hydrologic model development portion of this documentation, the subbasins were divided into three subareas based on whether a stormwater management facility controlled runoff and/or water quality on parcels. For the parcels controlled by a stormwater detention basin, an allowable discharge versus storage rating curve was created. Points on the rating curve were developed for the 2-year, 10-year, and 100-year storm events. The peak discharge from the detention basin was set to the pre-developed peak flow rate and the storage in the detention basin at that flow rate was calculated as the difference in volume from the existing runoff and the pre-developed runoff.

Hydraulic Model Development

The hydraulic model, HEC-RAS, was used to simulate the stream flow in the tributaries to Little Hunting Creek. The hydraulic model was specifically established to evaluate the following:

- Flood water overtopping at road crossings
- Drainage structure flooding
- Extent of predicted flooding
- Erosive velocities for selected design storms
- Benefits of LID and regional and onsite detention on the hydraulic conditions of the streams
- Optimal location of the peak shaving facilities

Three stream networks were modeled. These networks were the Paul Spring and North Branch streams, North Little Hunting Creek, and South Little Hunting Creek. Information that was gathered to develop the hydraulic model included the stream network, which was obtained from the county hydrography GIS dataset, cross section data points, which were developed from the county digital elevation data, and storm hydrographs, which were obtained from the hydrologic model.

A major part of the hydraulic model development was determining the location and number of cross sections. The cross sections for the model were located between 200 and 1,000 feet apart with a typical distance of 300 feet. However, multiple intermediate cross sections were interpolated, at 50-foot intervals, along the Paul Spring Branch and North Branch network. This step was necessary because of the abrupt variations in channel geometry that caused model instability. The characteristics of each cross section were established to include the channel roughness, the location of stream banks, and areas of ineffective flow. These parameters were established based upon county orthophotography, cross sections cut from the county DEM and field observations. The elevation of the cross section points was obtained from the county's digital terrain mapping, ArcView GIS, and the program HEC-GeoRAS.

Once the stream segments and cross sections were defined in the hydraulic model, the roadway crossing data was entered. Crossing information was collected in the field by survey crews and included the number and configuration of the stream culverts, details and configuration for bridges, roadway information, and culvert data such as diameter, material, and length. Ten crossings were modeled in the three stream systems. Six were on the North Branch and Paul Spring Branch system, three were on the North Little Hunting Creek system, and one was on the South Branch of Little Hunting Creek. The survey crossing data was compared to the county TIN data and the survey data was adjusted to match the elevations in the TIN.

The model was run for the 2-, 10-, and 100-year storm events. The results of the existing, future, and future proposed conditions were compared in order to determine the effect of the proposed alternatives on the stream stage and velocity.

Model Flow Verification

Once the initial hydrologic and hydraulic models were completed, the model output was compared to the water surface elevations and the depth of flow needed to overtop Paul Spring Road for known storm events. This information was used because no stream gauge data was available for Little Hunting Creek or its tributaries.

Residents noted that Paul Spring Road is frequently overtopped by flooding events. One resident provided specific dates for the storms that he had witnessed overtop the roadway: May 18 and July 15 of 2000 and June 12, June 14, and November 19 of 2003. On June 14, before the storm occurred in the afternoon, a photo was taken of the upstream end of the set of culverts showing debris that had lodged at the entrance due to previous storms. Though a storm occurred on June 14, there was no recorded rainfall for the event at the Washington National Airport rain gauge.

The flow hydrographs from subbasins LH-PS-0007, LH-PS-0006, and LH-PS-0005 in the SWMM model were entered into HEC-RAS for the storm event occurring June 12 through June 13. The peak flow for the June 12 storm for the initial parameter selection at Paul Spring Road was 68 cfs. The peak flow required to overtop the culverts at the roadway, if unobstructed, was 215 cfs, 3.5 times higher than the initial parameter selection, which corresponded to an approximate water surface elevation of 82.5 feet.

From a photo showing debris in front of the culvert and from residents' accounts that the culverts frequently become obstructed, it was determined that for the June 12 storm event that partially blocked culverts should be simulated. With the initial flow hydrographs, the HEC-RAS model was executed with one culvert blocked to a height of 3.95 feet at the upstream end of the culvert, the approximate height of debris that appeared in the June 14 photograph. Although the photograph does not visually show the obstruction to be completely blocking the barrel of the culvert, it was felt that the obstruction was significant enough to warrant almost complete obstruction.

Adjustments were made to the stormwater management controls for an upstream subarea. A review of the stormwater management subareas upstream of the Paul Spring Road culverts revealed that LH-PS-0007A, an area that was developed between 1972 and 1994, contained a land area 50.31 acres and a directly connected impervious surface area of 72 percent. The majority of this area is part of the Beacon Mall development, located on Richmond Highway across from Beacon Hill Road. Though the development date that was provided in the Little Hunting Creek parcel controls database was 1974, no surface stormwater management control was identified in the field for this parcel. Though underground detention may exist, it is suspected that if stormwater controls exist, they are either not functioning correctly or are inadequate for the size of the development and the high percentage of directly connected impervious surface area. The stormwater outfall drainage area that includes the Beacon Mall development is a double 60 inches x 60 inches box culvert. The SWMM model was adjusted to simulate uncontrolled flow from this parcel. The uncontrolled discharge from this outfall however did not produce the observed overtopping conditions, therefore several other parameters were adjusted as described below.

The directly connected impervious area was increased by 5 percent for all subareas in watershed to further increase the amount of runoff. A change was also made to the width parameter within the SWMM Runoff module by increasing it by 1.5 percent for all the subareas in the watershed. The maximum infiltration rate was adjusted by reducing WLMAX by 0.5 for all maximum infiltration values and by increasing the decay factor for the infiltration rate from an initial value for all subbasins of 0.0009 to 0.002.

Next, the model was run for a continuous simulation prior to the wet period. The dates were from June 1 to June 14. The infiltration regeneration coefficient, REGEN, was set to 0.01. Under these conditions, the model showed overtopping, but it was not stable going through the transition from culvert flow to culvert plus weir flow over the roadway. With the SWMM modeling parameters set within the range that was reasonable, the depth of debris in the second barrel of the culvert was set to 1.0 feet of debris. This was done to account for additional obstruction during the storm event from sediment and minor debris carried downstream. The increase in water surface elevation created by the additional stream flow depth stabilized the hydraulic model and produced the observed overtopping conditions. The final water surface elevation was 82.34 feet. The adjustment of the model parameters resulted in an average increase in peak flow for the subbasins in the watershed of 22 percent from the original flow values.

Proposed Alternatives

Woolpert developed alternative strategies to mitigate existing and potential stormwater related problems and to meet the goals and objectives for the watershed, which were developed by the Little Hunting Creek Steering Committee as part of the public involvement process. The alternative strategies were modeled within SWMM and HEC-RAS and the impacts were assessed.

Specific areas were identified for implementation of the recommended structural strategies and are displayed on Map 4.1. The structural practices that were modeled included the following:

- Retrofit of existing BMPs
- Construction of new BMPs
- Low-impact development (LID) zones for new and retrofit sites
- Wetlands
- Flow reduction in anticipation of Route 1 commercial corridor redevelopment
- Flow reduction for the Richmond Highway roadway widening project

Three methods were used to model the practices listed above: the BMP modeling method, the

LID modeling method, and the reduction in the percent flow method. Both new BMPs and retrofit BMPs were modeled using the BMP modeling method, which simulated the 1-year extended detention basin. For areas not already controlled for water quality, a percent pollutant removal efficiency was added to the BMP for water quality enhancements provided by the BMP. For the retrofit BMPs, the percent pollutant removal efficiency was based on the percentage of BMP coverage area.

The rain gardens, rain barrels, porous pavement, green roofs, and wetlands were modeled as LID facilities. Just as runoff from an area served by a biofiltration facility (rain garden) drains to a depression where it infiltrates into the ground, the hydrologic model directs flow to a node that is 100% pervious and infiltrates flow up to a maximum volume. After the maximum amount of infiltration is exceeded, the rainfall becomes runoff and flows to the storm drain system. The amount of flow needed to exceed the maximum infiltration was calculated to be the first half inch of runoff from the impervious surface in a subarea.

For the Route 1 planned commercial redevelopment area and for the anticipated Route 1 roadway widening project, a percent reduction in peak flow was calculated for the 10-year storm event. This was done by iteratively adjusting the percent impervious surface by the percent needed to reduce the percentage of flow. The percent reduction in flow for the Route 1 project was 5% and the percent reduction in flow for the commercial corridor was 10%.

After revising the model parameters in SWMM for the proposed BMPs, new hydrographs were created and the stream networks were modeled in HEC-RAS using the new hydrographs to evaluate changes in water surface elevation, floodplain limits and in-stream velocities.

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