Section 4: Model Development and Application

The hydrologic model for the Wissahickon Act 167 study was built using GIS layers of land use, hydrologic soil groups, terrain and orthophotography. Within the Sandy Run watershed, model sub-watershed boundaries match those used for the hydrologic and hydraulic modeling completed for the recent flood insurance study of the suburban Sandy Run Watershed. The modeling for this portion of the watershed was coordinated with an updated flood insurance study and the hydrology has been approved by the Federal Emergency Management Agency (FEMA). The model has been calibrated against two U.S. Geological Survey stream gaging stations located on the Wissahickon Creek at Fort Washington and at Philadelphia.

4.1 Development of the Act 167 Hydrologic Model

The objectives of the Act 167 hydrologic modeling were: to determine peak rate controls for stormwater management; to assess the hydrologic impact of potential land use change; to obtain flows at obstructions, and to evaluate the potential impacts of stormwater improvements.

Subbasin delineations were chosen primarily at stream confluence points and boundary delineations were based on several sources. These included a digital elevation model (DEM) and 2-foot contour interval data obtained by the Center for Sustainable Communities (CSC), 2-foot contour data provided by PWD, and, particularly within the city limits of Philadelphia, storm sewer shed delineations provided by PWD. The Wissahickon Watershed was subdivided into 137 subbasins as shown in Figure 4.1.A.

The outer boundary shown with the modeling results in this report is slightly different from that shown for the informational maps in Sections 2 and 3 and the peak rate management district maps in Section 5 (Figure 5.3.A) and the Ordinance (Appendix A). The reason for this is that the model boundary was set up to precisely match with previous flood insurance study modeling of the Sandy Run portion of the watershed. Because the outer boundary in Figure 5.3.A and the Ordinance conforms more closely with the neighboring Pennypack watershed boundary and with boundaries in use by the Philadelphia Water Department and PADEP, it is recommended that it be used as a guide for the purpose of peak rate management. The difference does not affect the peak rate district modeling results. The precise determination of the applicable peak rate district for individual projects near the boundary would be determined form site plans as specified in Section 408.C of the Ordinance.

Land Use Data for 2005 from the Delaware Valley Regional Planning Commission (DVRPC) and NRCS data for Hydrologic Soil Groupings were used to generate NRCS runoff Curve Numbers for each of the 137 subbasins. Figure 4.1.B shows the distribution of runoff Curve Numbers calculated for the Wissahickon Watershed. These are composite Curve Number values that include the effect of impervious cover, such as roof and parking areas, as well as pervious areas.

In addition to the volume of precipitation that runs off the land surface, the shape and slope of each subbasin affect the timing of the runoff and the peak flow. For this study, these factors are represented by the subbasin time of concentration (Tc), which was calculated as the sum of sheet flow time, shallow concentrated flow time, and channel flow time for the longest flow path to the subbasin outlet. Orthophotography, elevation contours and digital elevation models (DEMs) were used to estimate slope and the length of each flow path. The maximum length of sheet flow was limited to 100 feet.[1]

The Act 167 hydrologic model also includes 87 stream reaches to convey flow from the subbasin outlets through the tributaries and main stem of the Wissahickon Creek. Flow rates through reaches are influenced by storage defined by the shape of the channel and over banks and by the friction generated. This relationship was used to apply Muskingum-Cunge routing to the stream reaches in the model. This method represents the reach using channel length, an average cross section, and Manning's roughness coefficients. Figure 4.1.C shows a sample schematization of stream reaches. A Type II rainfall distribution developed for interior portions of the continental United States was used for modeling the storm events.

The Act 167 model parameters for subbasins and reaches are provided in Appendix B.

[1] *Merkel, References on Time of Concentration with Respect to Sheet Flow, National Water and Climate Center, 2001.*

Figure 4.1.A Subwatershed Delineation for the Wissahickon Watershed Showing 137 Subbasins.

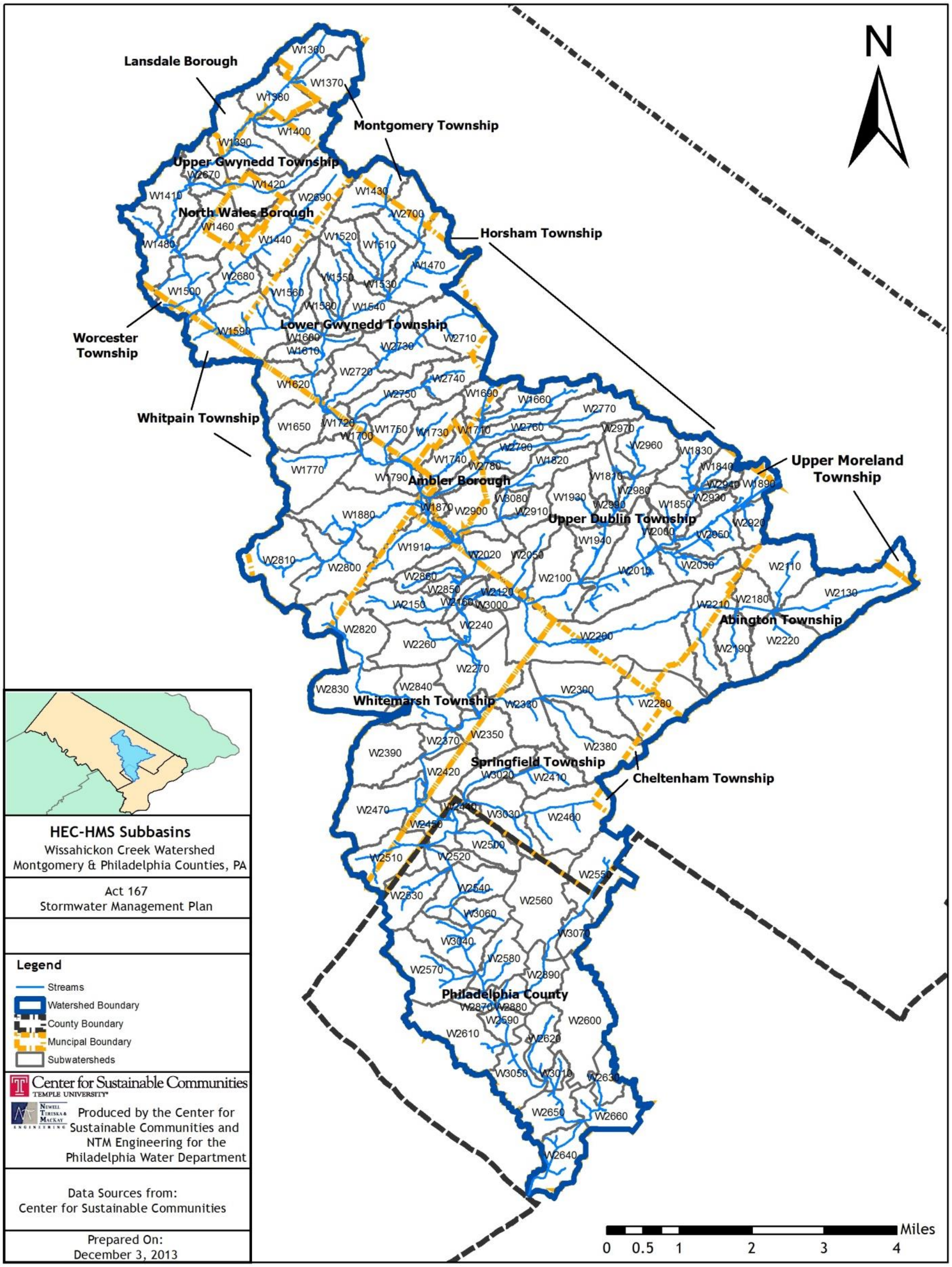


Figure 4.1.B NRCS Runoff Curve Numbers for Each of the 137 Subbasins

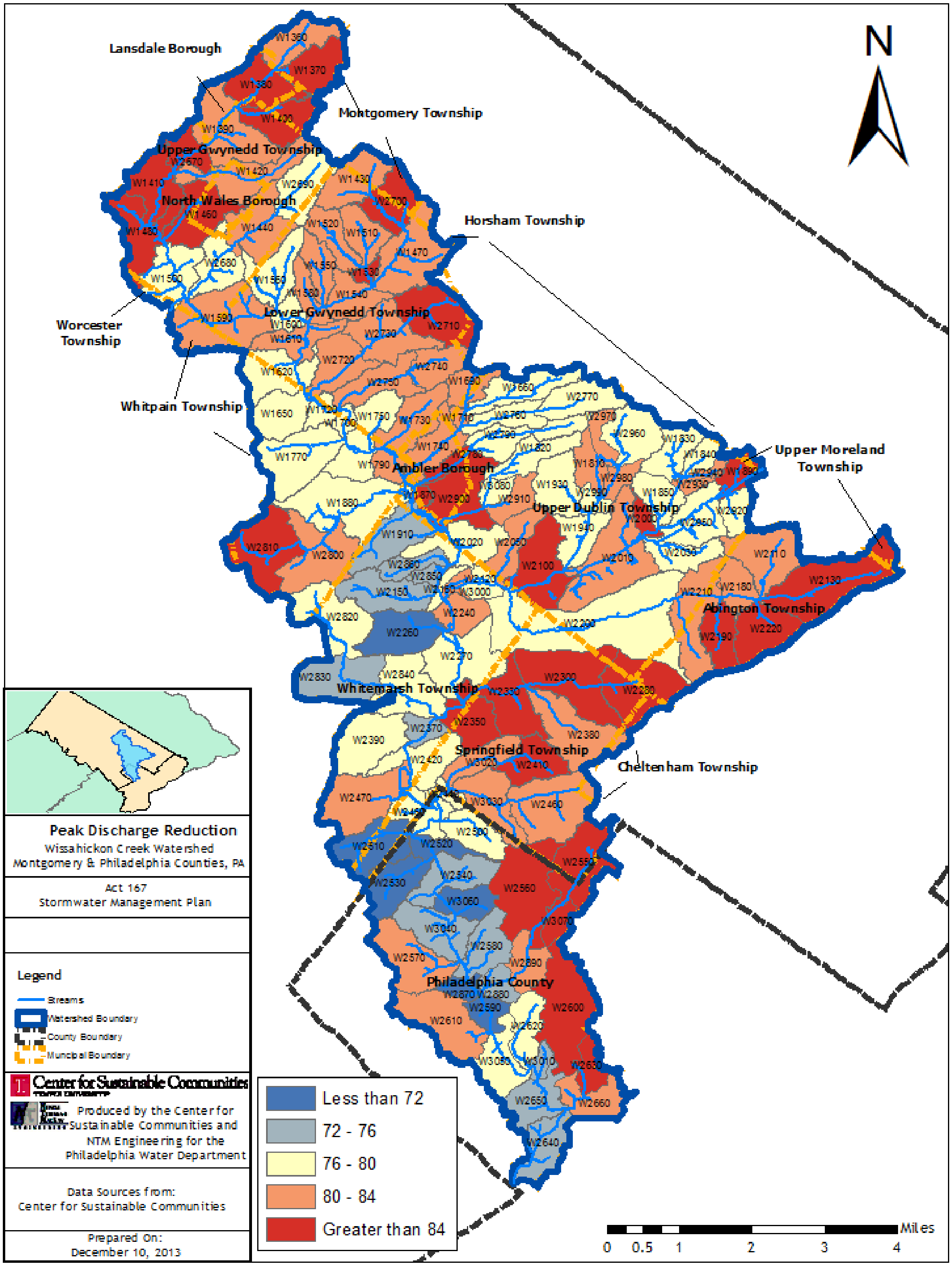
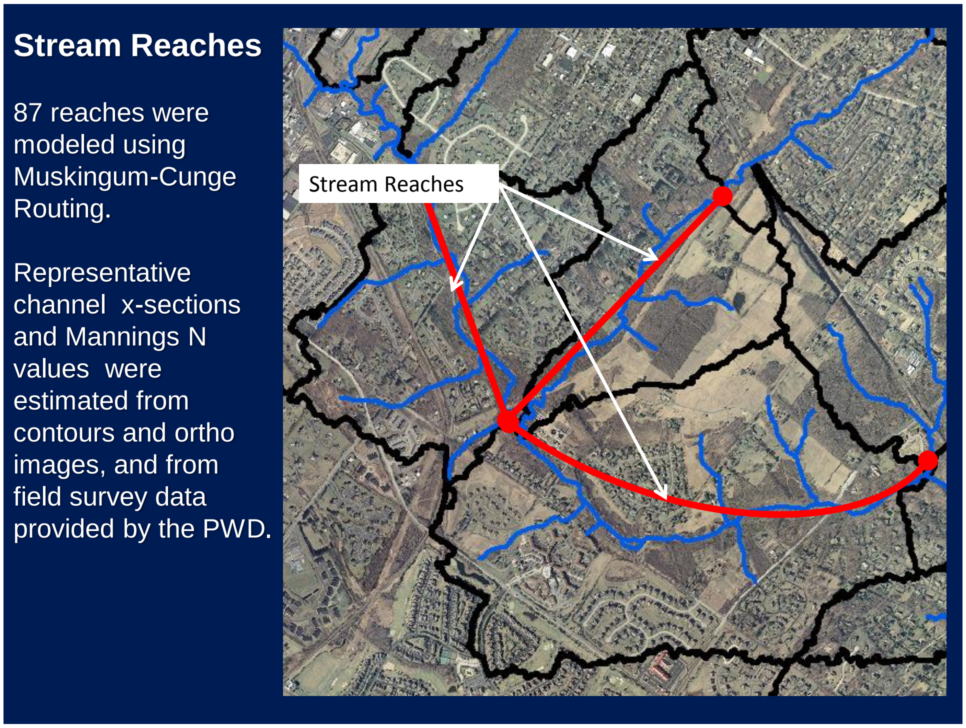


Figure 4.1.C Sample Stream Reach Schematization



4.2 Modeling Assumptions

Assumptions included in the hydrologic modeling affect the representation of the rainfall-runoff process and the potential applications of the model. The key modeling assumptions include:

- Subbasin properties are averaged for each subbasin area. Subbasin areas ranged from 0.06 to 2.11 square miles and averaged 0.46 square miles.
- The hydrologic impact of stormwater piping is not included in the modeling. The Tc for the subbasins was calculated based on surface features.
- For the design events, the same total volume and temporal distribution of rainfall is applied over each of the subbasins.
- Design storm precipitation totals were obtained from PennDOT's PDT-IDF curves based on NOAA Atlas 14.
- Design storm precipitation timing was assigned a Type II distribution, which concentrates most of the rainfall during the middle portion of the storm event.
- The maximum distance for sheet flow was assumed to be 100 feet.
- Curve Numbers for each subbasin were calculated so that the impervious cover associated with each land use type was included.
- Facilities with storage volumes larger than 5 Acre-Feet were modeled as reservoirs. For smaller facilities, the aggregate total of detention storage in each subbasin was considered additional potential storage. The Curve Number for each subbasin was adjusted downward to account for this using the NRCS Curve Number equation.
- Tc was calculated as the sum of sheet flow, shallow concentrated flow and channel flow. Subbasin lag was calculated as 0.6*Tc.

It is important to note that the scale of the model, while considered adequate for purposes of the Act 167 study, is not suitable for site level analysis or design.

4.3 Model Calibration

The hydrologic model developed for the Act 167 Plan included 137 subbasins for the 64-square mile Wissahickon Watershed. A schematic diagram of the model is shown in Figure 4.3.A. The model was based on Natural Resource Conservation Service (NRCS) Curve Number and Unit Hydrograph procedures available within the HEC-HMS modeling software and was developed by Temple University's Center for Sustainable Communities and calibrated by NTM Engineering following the general procedure outlined in Figure 4.3.B and detailed in the calibration report which is available upon request.

Figure 4.3.A   Diagram of Wissahickon 137-Subbasin Hydrologic Model

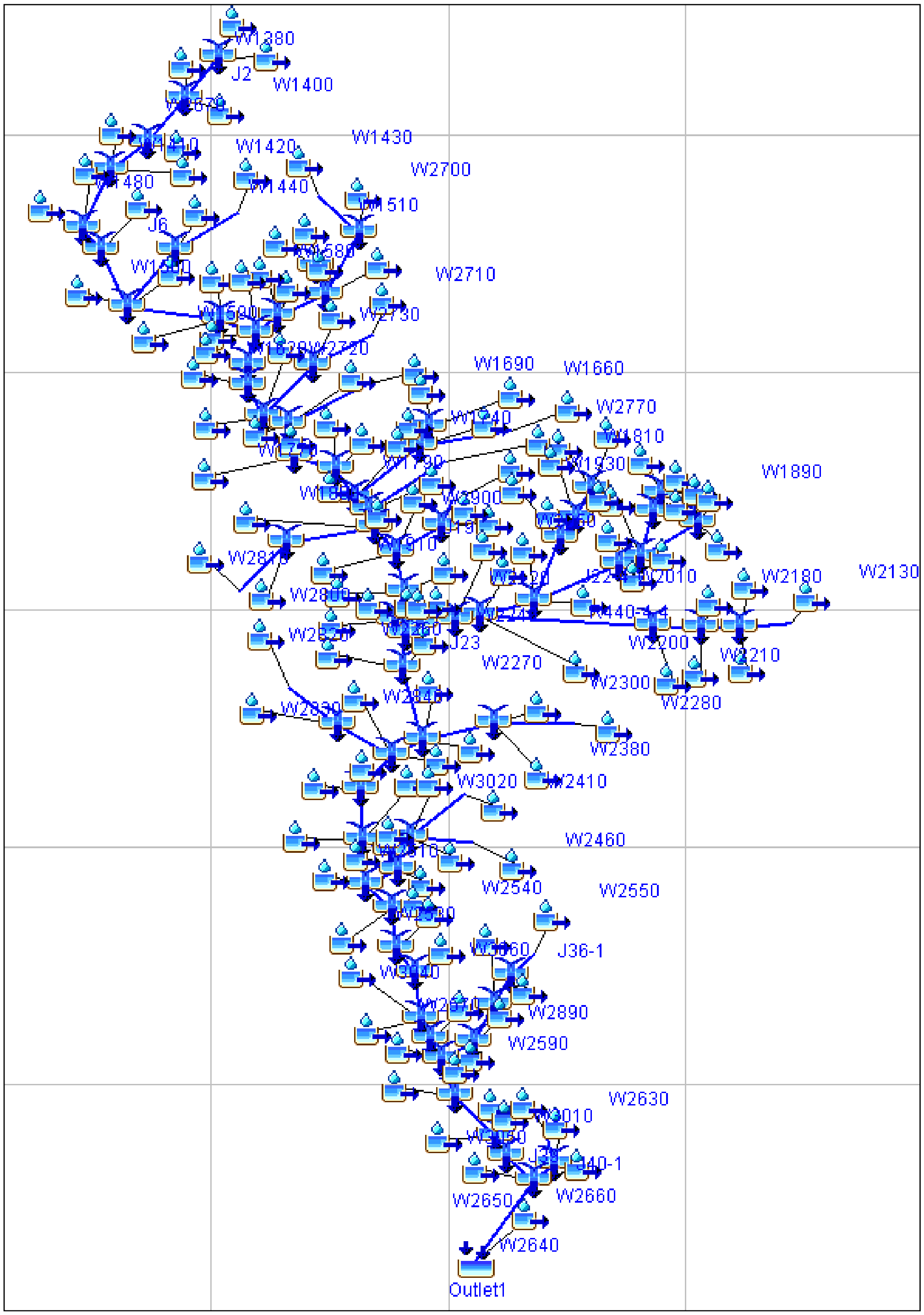
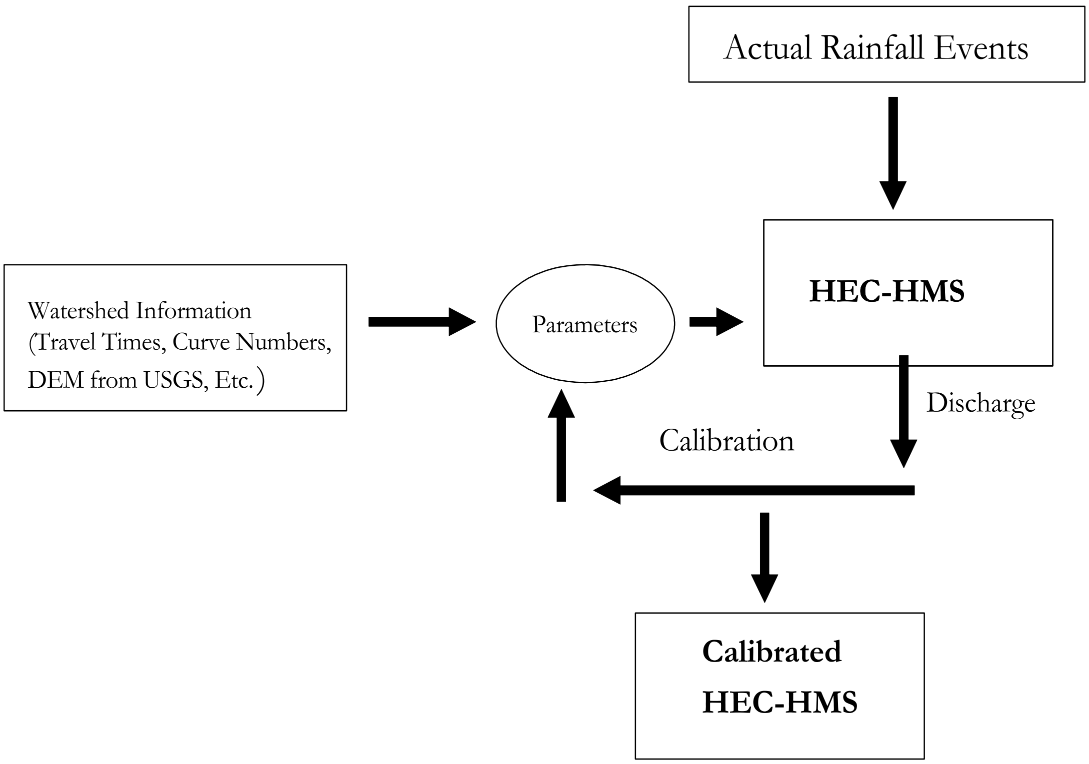


Figure 4.3.B Development and Calibration of the 137-Subbasin Hydrologic Model



4.4 Model Applications

The hydrologic model was applied to several components of the study. Each of the applications is summarized in this section.

- Evaluation of hydrologic impacts of land use change
- Determination of peak flow rates for identifying frequently flooded bridges and culverts
- Determination of peak rate control management districts included in the model ordinance
- Evaluation of runoff impacts of improved stormwater control through BMP applications

Evaluation of the Hydrologic Impacts of Future Land Use Change

Details of the two land use scenarios developed for this study are presented in Section 2.3. For each scenario, the projected land use was used to calculate revised curve numbers for each model subbasin. The new curve numbers were then used in the model to determine runoff volume and peak flow rates. The results were compared to existing conditions to determine the magnitude of the changes. A summary of the results is provided below.

<u>Trend Scenario</u>
This scenario projects that land use demand from Year 2040 projected population growth will be met under primarily through the use of available, non-restricted open space with suitable terrain.

The method used for this projection is described in Section 2.3. The purpose of the model run was to determine the additional runoff volume due to projected land use change only, in the absence of stormwater controls. While this model run does not include the stormwater measures that would normally be required for new development, it provides a measure of the runoff volume increase that could be expected in the Wissahickon Watershed in the absence of sustainable land use practices. For this scenario, runoff volume for the Wissahickon Creek at the mouth in Philadelphia increases by 1 percent during the 1-Yr storm event and increases by between 0.4 and 0.7 percent for larger storm events as shown in Figure 4.4.B. This small increase in volume reflects the slow rate of growth projected for the watershed as a whole. Subbasin runoff increases during the 1-Yr storm event are shown in Figure 4.4.A. Comparison with projected land use in Figure 2.3.A shows that the subbasins with the largest runoff volume increases are located on areas where significant new residential or non-residential land use is projected.

<u>Green Scenario</u>

The Green Scenario assumes that Year 2040 population growth can be met using higher density residential and non-residential development, with a focus on redevelopment and use of "stacked" or cluster and mixed land uses. The details of the assumptions for meeting land use demand are presented in Section 2.3. The purpose of the model run was to determine the reductions in runoff that this approach could offer versus the Trend Scenario. As with the Trend Scenario, the model run was performed without stormwater control measures in order to isolate the impact of the land use approach. The analysis showed that under the assumptions applied, runoff curve numbers were actually reduced slightly for a number of subbasins due to the use of developed land (already with a high curve number) to accommodate growth. Figure 4.4.B compares the aggregate runoff impact of the Green Scenario versus the Trend Scenario and Existing Condition. The results show that at the mouth of the Wissahickon Watershed runoff volume is reduced by 2 percent over the Trend Scenario and 1 percent over existing conditions. While these differences are small, the result supports the concept that land use management based on suitability criteria offers a means of control for future runoff volume that supplements the use of extended detention and other BMPs. In practice, site-by-site analysis of development or re-development is required to fully evaluate runoff impacts.

<u>Future Conditions Evaluation</u>

This study also included a model analysis of "Future Conditions" to evaluate the runoff impacts of potential stormwater control measures such as detention basin retrofits, infiltration, and restoration of riparian buffer areas. The Green Scenario was used to represent the future land use condition for this evaluation. This was done to account for changes in land use practices toward mixed use and clustering to accommodate new demand, given the highly developed status of the Wissahickon Watershed and the potential application of more sustainable land use practices. The results of the Future Conditions modeling are summarized at the end of this section.

Figure 4.4.A Volume Increase for 1-Yr Storm – Trend Scenario

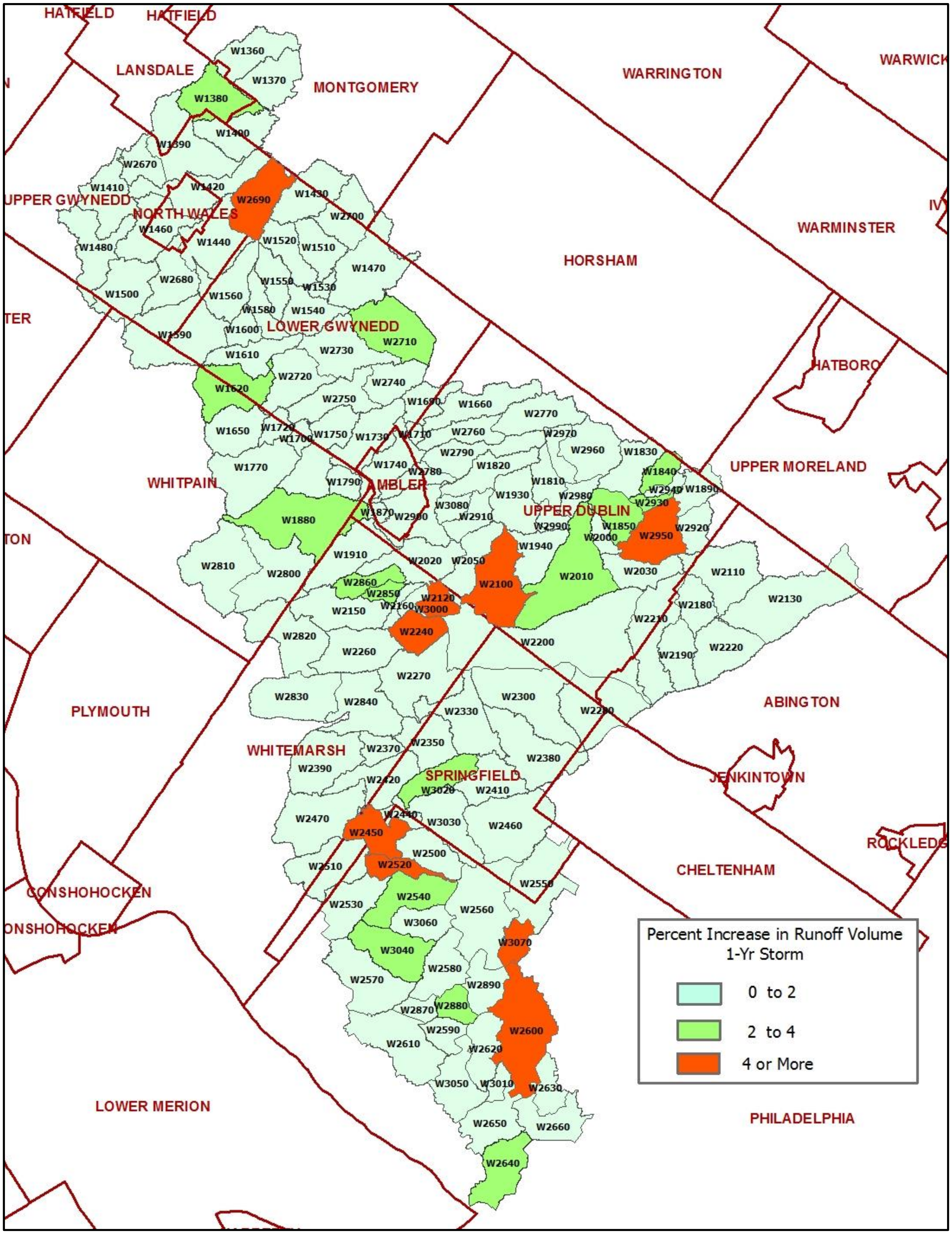
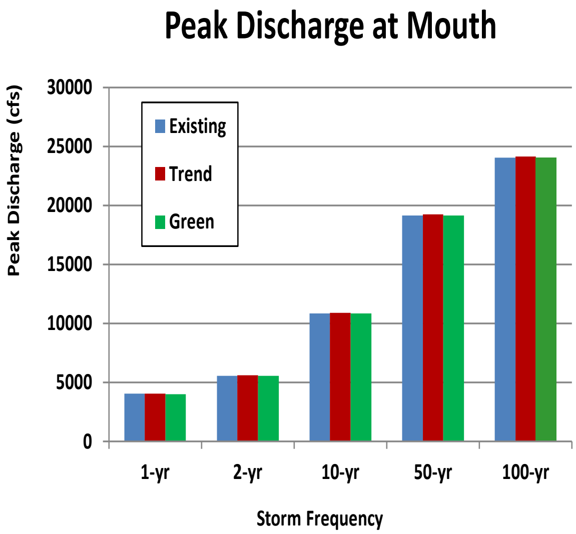
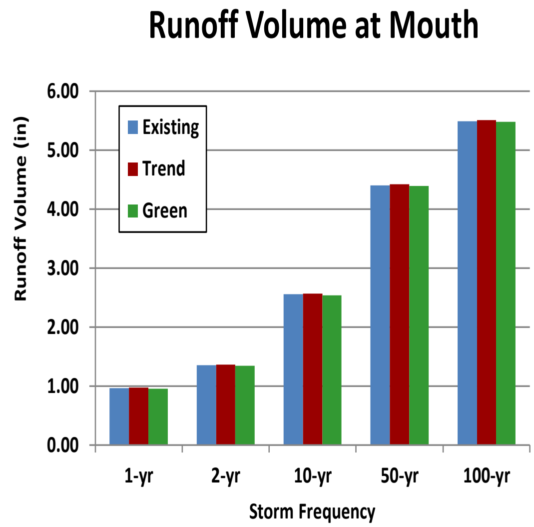


Figure 4.4.B Hydrologic Impact of Projections for Wissahickon Creek at Mouth

Existing vs. Trend vs. Green Modeling Comparison





Peak Discharge at Mouth (cfs)

Storm	Existing	Trend	Green	% Change Trend	% Change Green
1-yr	4043	4066	4024	0.6	-0.4
2-yr	5591	5627	5568	0.6	-0.4
10-yr	10871	10925	10847	0.5	-0.2
50-yr	19148	19224	19121	0.4	-0.1
100-yr	24057	24136	24029	0.3	-0.1

Volume at Mouth (in)

Storm	Existing	Trend	Green	% Change Trend	% Change Green
1-yr	0.97	0.98	0.96	1.0	-1.0
2-yr	1.36	1.37	1.35	0.7	-0.7
10-yr	2.56	2.57	2.54	0.4	-0.8
50-yr	4.40	4.42	4.39	0.5	-0.2
100-yr	5.49	5.51	5.48	0.4	-0.2

Determination of Peak Flow Rates for Identifying Flood-Prone Bridges and Culverts

The hydrologic model was applied to determine obstructions (bridges and culverts) where capacities are most likely to be exceeded by flooding. PWD provided the CSC and NTM Engineering with a GIS shape file including 370 bridges and culverts located in the Wissahickon Watershed, that were identified as significant obstructions to flow. The PWD and the CSC preformed field measurements to update the dimensions of these structures. The study team then calculated the full flow capacity of most of these structures, accounting for the potential headwater upstream of the structure. The hydrologic model was used to calculate peak discharges at each obstruction for the 1-year, 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year storm events. The discharges were then compared to the calculated capacity of each obstruction. The results of the comparison were presented in Figure 3.1.E as part of the description of flood problems in the watershed. The method applied does not take debris blockage or potential downstream submergence into account, but provides a screening tool to identify structures where the free flow capacity to convey flooding is most limited.

Determination of Peak Rate Control Management Districts Included in the Model Ordinance

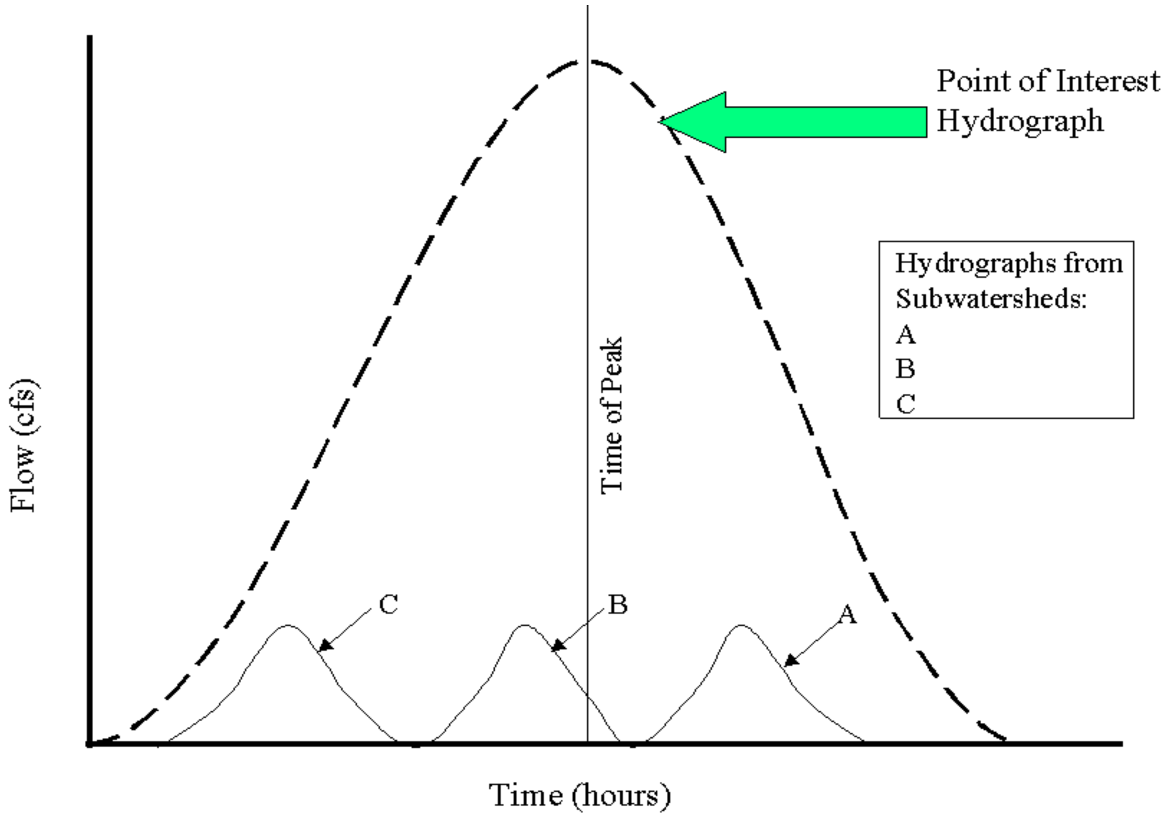
Stormwater management criteria include peak rate control in order to prevent post development flood discharge from exceeding pre-development discharge and worsening downstream flooding. Because detention basins used to control increased peak flows and runoff volumes from development also slow the timing of outflow, an understanding of runoff timing throughout the watershed is needed to establish peak rate criteria. Under some conditions, delaying runoff at a site can cause the peak from the site to better coincide with the peak from other parts of the watershed at downstream locations. This may occur even when the detention basin limits outflow so that there is no increase in the runoff rate from the site after development. This can worsen downstream flooding and increase erosion for a given storm. Because it accounts for the timing of flow through the subbasins and stream reaches, a hydrologic model is useful for defining post-development runoff rates that will prevent this situation from occurring.

The objective of modeling for peak rate control is to determine the flow contribution of different subareas in the watershed (model subbasins) to the peak discharge at various locations downstream, and then determine which subbasins can potentially worsen flooding at the downstream location if runoff is detained. The method follows the procedures presented by DeBarry for establishing stormwater management districts.[2] This analysis establishes "Points of Interest" and evaluates the effect of lagged hydrographs from portions of the watershed upstream to these points of interest as shown in Figure 4.4.C. The 50-year storm event was used in the modeling to determine routing time and flow contributions for a Type II storm event. The time required for discharge from each upstream subbasin to reach a given point of interest was determined on order to "lag" the subbasin hydrograph, and see how it actually contributes to the peak flow at the point of interest as it flows past the location. If the lagged peak flow from the subbasin occurs after the peak flow at the point of interest, then detention in that subbasin would not worsen flooding at that location as shown for Case A in Figure 4.4.C If it occurs before the peak, particularly as in Case B, detention can worsen flooding and a peak rate control is necessary to protect the point of interest. In general, for the Wissahickon Watershed, headwater subbasins

[2] *DeBarry, P.A., Watersheds, Processes, Assessment, and Management, John Wiley & Sons, Inc., 2004, Section 18.5.*

fall into the first category, while subbasins in the middle portions of the watershed fall into the second group.

Figure 4.4.C Points of Interest Method Used for Modeling to Determine Release Rates[3]



For subbasins where detention could worsen flooding, the ratio of the contributing discharge at the time of peak flow at the point of interest, to the peak flow of the subbasin, is taken as the "release rate" and can be expressed as a percentage. For example, a release rate of 70 percent means that the lagged subbasin flow at the time of the peak discharge at the point of interest is 70 percent of the subbasin peak flow. To prevent worsened flooding at the point of interest, detention to control new runoff volume should limit discharge to 70 percent of the pre-development peak. Release rates for all upstream subbasins were calculated for each point of interest shown in Figure 4.4.D, and the minimum release rate for each subbasin was then determined.

The release rates determined using the model were used to establish where rate controls should be applied to prevent detention at new development sites from increasing flood flows. The calculated release rates were then used to establish the stormwater management districts shown in Figure 4.4.E. These management districts are incorporated with the recommended stormwater management criteria in Section 5, and with the Ordinance in Appendix A.

[3] *DeBarry, P.A., Watersheds, Processes, Assessment, and Management, John Wiley & Sons, Inc., 2004, Figure 18.4.*

Figure 4.4.D Points of Interest Used for Modeling to Determine Release Rates

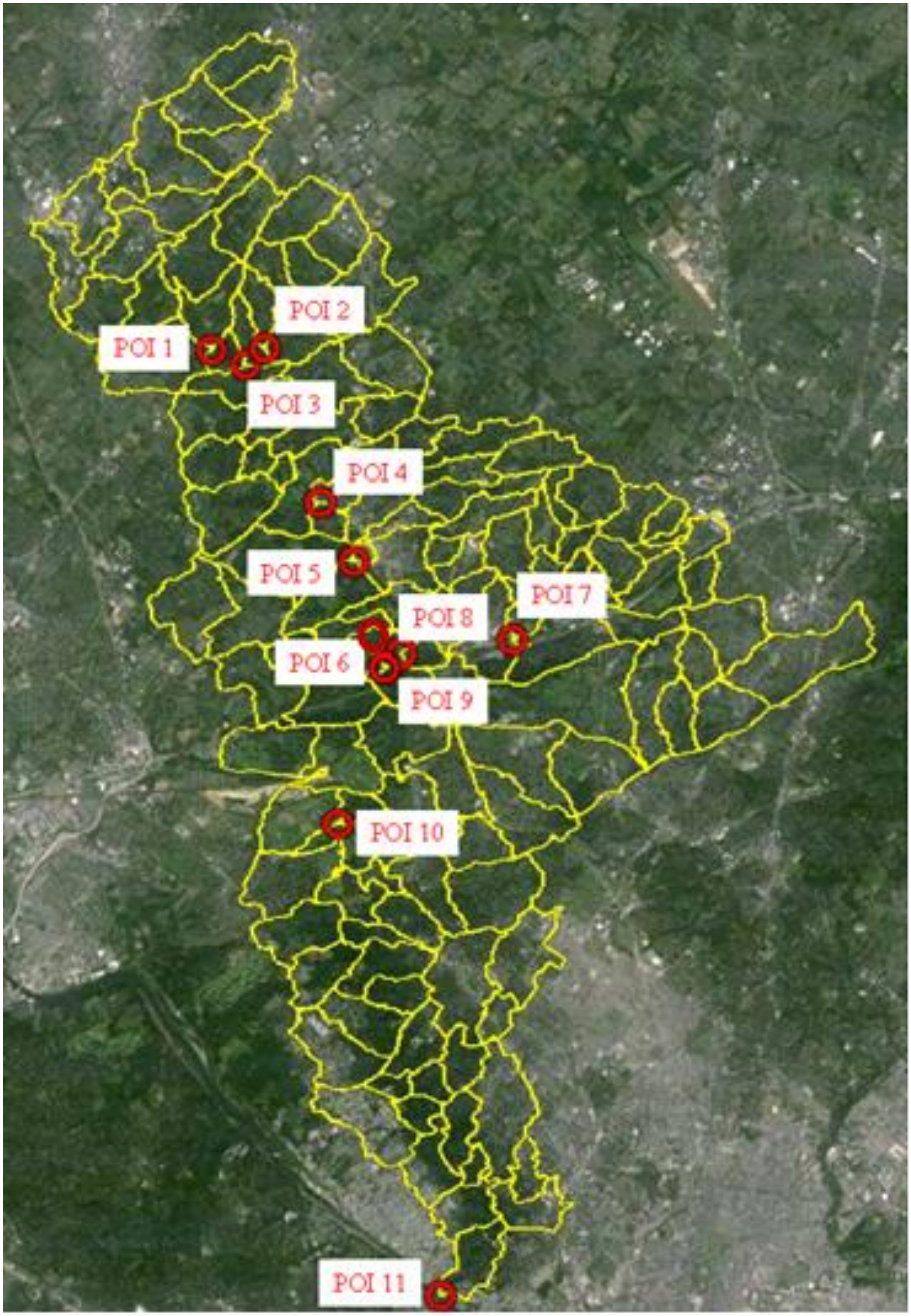
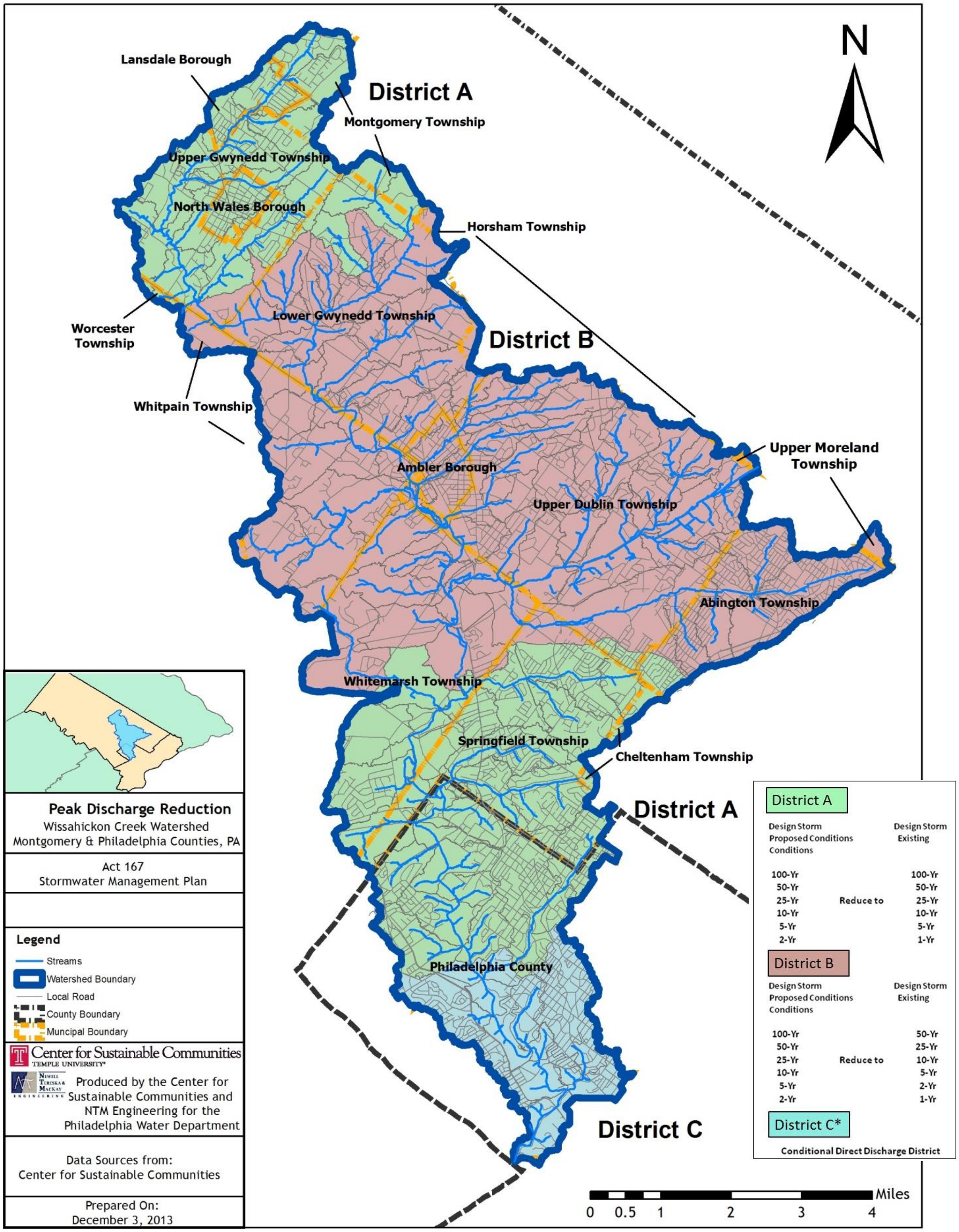


Figure 4.4.E Peak Rate Management District Map for the Wissahickon Watershed



Evaluation of Runoff Impacts of Improved Stormwater Control through BMP Applications

The hydrologic model was applied to a "Future Conditions" analysis to evaluate the hydrologic impact of implementing identified opportunities for installation and/or retrofitting of stormwater BMPs. These potential improvements are presented in Section 6 of this report. Three categories of BMP applications were considered: new or expanded detention, infiltration, and restoration of riparian buffers along stream corridors. For the modeling of potential stormwater improvements, several of the larger facilities were individually modeled in HEC-HMS as reservoirs, as were existing facilities with storage capacity exceeding 5 acre-ft.

Modeling of the proposed facilities included two new flood retarding structures on Rapp Run and Pine Run in the Sandy Run portion of the watershed. Design details for these structures were provided by URS, Inc. at the request of Upper Dublin Township. These on-stream structures are equipped with openings to allow passage of normal stream flows up to near bank-full stage. The structures provide a combined total of 270 acre-ft of flood storage at their spillway crests. These structures operate differently than off-stream extended detention facilities, which are equipped with outlet structures designed to retain runoff from small storms.

For the smaller detention facilities, the total storage was considered additional potential storage available during the course of a given storm event, and the Curve Number for the subbasin was adjusted downward using the NRCS Curve Number equation.[4] The total additional infiltration storage in each subbasin was modeled as initial abstraction, with one inch of storage assumed for most of the site areas. Restored riparian buffer acreage was also assumed to provide one inch of additional storage over the restored acreage and was modeled as initial abstraction. Projected land use for this model run was represented by the "Green Scenario" presented in the land use modeling discussion.

For existing conditions modeling, existing detention facilities were modeled in the same manner as for the future conditions run, with individual modeling of facilities with more than 5 acre-ft of storage capacity.

Details of the proposed improvements and their hydrologic impacts are discussed in Section 6.

The calibrated model developed for this study produces peak flow rates for the Sandy Run watershed that are lower than previous modeling by Temple University for its Fort Washington Area Study in 2008. The 2008 model was developed for the Sandy Run Watershed only and was not calibrated against observed data due to the lack of a stream gage in the watershed. Because the model for this study was developed for the entire Wissahickon Watershed, the calibration process utilized the only available stream gages on the Wissahickon Creek at Fort Washington and Philadelphia. For tributary areas in Sandy Run, the results for this model compare well with peak flow values calculated using Regional Regression equations developed by the U.S. Geological Survey.[5] Flow and precipitation measurement within the Sandy Run and other densely developed watershed would provide a bases for further model testing and frequency analysis.

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[4] *Urban Hydrology for Small Watersheds, TR55, Natural Resources Conservation Service, 1986*

[5] Roland, M.A., and Stuckey, M.H., 2008, Regression equations for estimating flood flows at selected recurrence intervals for ungaged streams in Pennsylvania: U.S. Geological Survey Scientific Investigations Report 2008-5102