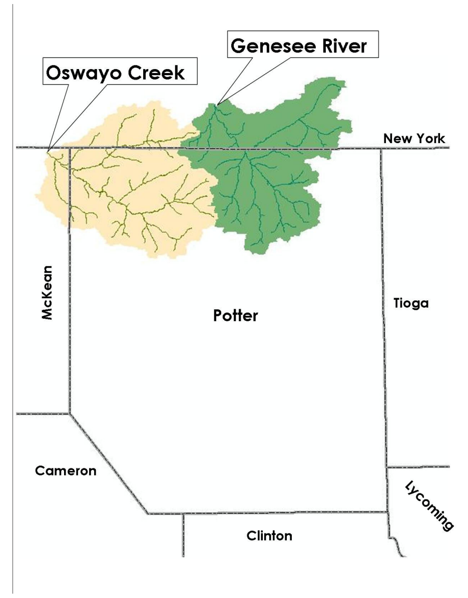


Appendix A – Watershed Modeling Technical Data

An overview of the process that was used to complete the hydrologic modeling in preparation of this Plan is presented in *Section 6 – Technical Analysis* of this report. The following technical data is included here to supplement the general information provided in that section.

DATA COLLECTION

The GIS data for the hydrologic models was compiled from a variety of sources by county, state, and federal agencies. The data was collected in and processed using GIS software. A description of GIS data collected, the source and its use is provided in *Table A.1*.

Data	Source	Use
10-m Digital Elevation Model (DEMs)	USGS (2008a)	Watershed delineation, length, basin slope, stream slope, average elevation
High Resolution Streamlines	USGS (2008b)	Watershed delineation, cartography, spatial orientation
National Land Cover Dataset – Land Use 2001	USGS (2008c)	Curve number generation for watershed subareas outside of county boundary
Existing land use for year 2005	Potter County GIS	Curve number generation for watershed subareas for year 2010 within county boundary
Future land use for year 2015	Potter County GIS	Curve number generation for watershed subareas for year 2020 within county boundary
SURRGO Soils Data	NRCS (2008)	Curve number generation; analysis of infiltration limitations
Storage (percent of lakes, ponds, and wetlands)	USGS (2008d)	Calculation of parameters for USGS Regression Equations
Roadway Data	Potter County GIS	Cartography, spatial orientation

Table A.1. GIS Data Used in Act 167 Technical Analysis

HYDROLOGIC MODEL PARAMETER DATA

SOILS, LAND USE, AND CURVE NUMBERS

The determination of curve numbers is a function of soil type and land use. The hydrologic soil groups were defined by NRCS (2008). The 2001 NLCD was simplified to provide an estimate of curve numbers using the scheme shown in *Table A.2*.

GIS Value	NLCD (2001) and Potter County Description	NRCS (1986) Description	A	B	C	D
11	Open Water	Water	98.0	98.0	98.0	98.0
21	Developed, Open Space	Open space - Good Condition	39.0	61.0	74.0	80.0
22	Developed, Low Intensity	Residential - 1 acre	51.0	68.0	79.0	84.0
23	Developed, Medium Intensity	Residential - 1/2 acre	54.0	70.0	80.0	85.0
24	Developed, High Intensity	Commercial and Business	89.0	92.0	94.0	95.0
31	Barren Land (Rock/Sand/Clay)	Newly graded areas	77.0	86.0	91.0	94.0
41	Deciduous Forest	Woods - Good Condition	30.0	55.0	70.0	77.0
42	Evergreen Forest	Woods - Good Condition	30.0	55.0	70.0	77.0
43	Mixed Forest	Woods - Good Condition	30.0	55.0	70.0	77.0
52	Shrub/Scrub	Brush - Good Condition	30.0	48.0	65.0	73.0
71	Grassland/Herbaceous	Meadow - Good Condition	30.0	58.0	71.0	78.0
81	Pasture/Hay	Pasture - Good Condition	39.0	61.0	74.0	80.0
82	Cultivated Crops	Contoured Row Crops - Good Condition	65.0	75.0	82.0	86.0
90	Woody Wetlands	Woods - Good Condition	30.0	55.0	70.0	77.0
95	Emergent Herbaceous Wetlands	Water	98.0	98.0	98.0	98.0
110	Residential	Residential - 1/2 acre	54.0	70.0	80.0	85.0
111	Residential (High Density or Multiple Dwelling)	Residential - 1/8 acre	77.0	85.0	90.0	92.0
114	Residential (Rural, Single Unit)	Residential - 1 acre	51.0	68.0	79.0	84.0
120	Commercial and Services	Commercial and Business	89.0	92.0	94.0	95.0
123	Other Commercial	Commercial and Business	89.0	92.0	94.0	95.0
130	Industrial	Industrial	81.0	88.0	91.0	93.0
180	Recreation Land	Woods - Good Condition	30.0	55.0	70.0	77.0
190	Recreation Open Use	Open space - Good Condition	39.0	61.0	74.0	80.0
210	Cropland/Pasture	Pasture - Good Condition	39.0	61.0	74.0	80.0
213	Idle Fields	Pasture - Good Condition	39.0	61.0	74.0	80.0
220	Orchard/Nursuries/Horticulture	Contoured Row Crops - Good Condition	65.0	75.0	82.0	86.0
240	Other Agriculture	Contoured Row Crops - Good Condition	30.0	55.0	70.0	77.0
320	Upland Shrubs	Woods - Good Condition	30.0	55.0	70.0	77.0
330	Mixed Range Land	Pasture - Good Condition	39.0	61.0	74.0	80.0
410	Deciduous Forest	Woods - Good Condition	30.0	55.0	70.0	77.0
413	Aspen-Birch Forests	Woods - Good Condition	30.0	55.0	70.0	77.0
420	Confierous Forest	Woods - Good Condition	30.0	55.0	70.0	77.0
440	Brushland/Shrubland	Woods - Good Condition	30.0	55.0	70.0	77.0
510	Waterways/Streams/Canals	Water	98.0	98.0	98.0	98.0
530	Artificial Lakes	Water	98.0	98.0	98.0	98.0
1000	Village	Residential - 1/8 acre	77.0	85.0	90.0	92.0
1001	Rural Growth	Residential - 1 acre	51.0	68.0	79.0	84.0
1002	Rural Hamlet	Residential - 1/2 acre	54.0	70.0	80.0	85.0
1003	High Growth	Commercial and Business	89.0	92.0	94.0	95.0

Table A.2. Existing Curve Number Determination Potter County and Outside Areas for each Hydrologic Soil Group

The curve numbers presented in the above tables represent "average" antecedent runoff condition (i.e. ARC = 2). In a significant hydrologic event, runoff is often influenced by external factors such as extremely dry antecedent runoff conditions (ARC=1) or wet antecedent runoff conditions (ARC=3). The antecedent runoff conditions of the above curve numbers were altered during the calibration process so that model results are within a reasonable range of other hydrologic estimates.

HYDROLOGIC MODEL PREPARATION

Two Act 167 designated watersheds within the county were selected for hydrologic modeling: Oswayo Creek and Genesee River. These watersheds were delineated into subwatersheds based on problem areas, significant obstructions, and natural subwatershed divides. The delineation of these subwatershed areas created points of interest at junctions where the subwatersheds were hydraulically connected in the HEC-HMS model.

OSWAYO CREEK MODEL

The Oswayo Creek watershed has a drainage area of 182.1 square miles and was divided into 96 subbasins for the HEC-HMS model. Approximately 61 square miles of this watershed are located outside of Potter County. *Figure A.1* illustrates the Oswayo Creek subwatersheds and cumulative discharge points.

This watershed does not contain any dams that were considered to have a significant enough impact on the hydrology of the watershed. For this study, dams with small storage volumes (less than 100 acre-feet) and dams that would be completely filled during minor runoff events (0.3 inches of runoff) were generally considered "run-of-the-river dams" that would only affect the immediate area near the dam. Their impacts to the overall watershed hydrology within Potter County were considered negligible and were not included in this study.

GENESEE RIVER MODEL

The Genesee River watershed within Potter County has a total drainage area of approximately 97 square miles. The modeled watershed included small parts of the Genesee River that are located outside of Potter County. The entire modeled watershed has a total drainage area of approximately 166.6 square miles and was divided into 82 subbasins for the HEC-HMS model. *Figure A.2* illustrates the Genesee River subwatersheds and cumulative discharge points.

This watershed does not contain any dams that were considered to have a significant enough impact on the hydrology of the watershed. The "run-of-the-river dam" criteria as discussed above for deciding whether or not a dam would be included was also used for the Genesee River within Potter County.

FIGURE A.1

FIGURE A.2

HYDROLOGIC MODEL PARAMETERS

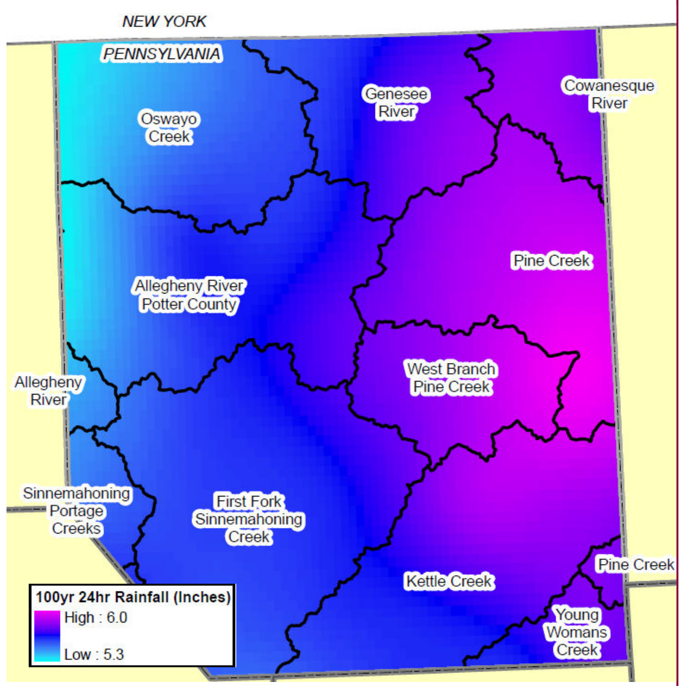
The various parameters entered into the hydrologic models include subwatershed area, soil-type, land cover, lag time, reach lengths and slopes, reach cross sectional dimensions, and design rainfall depths. A brief description of these components follows.

RAINFALL DATA

Rainfall data used in this modeling effort incorporates rainfall runoff data from the NOAA Atlas 14. NOAA Atlas 14 provides the most up to date precipitation frequency estimates, with associated confidence limits, for the United States and is accompanied by additional information such as temporal distributions and seasonality. The following table provides the rainfall estimates used for various design storm frequencies for Potter County (NOAA, 2008):

Design Storm (years)	Design Depth (in)
2	2.46
10	3.52
25	4.29
50	4.99
100	5.81

Table A.5. Rainfall Values for Potter County



NOAA Atlas 14 100-year, 24-hour Rainfall

It was assumed in all of the following analyses that these single rainfall quantities could be applied uniformly over the entire subwatershed area. Additionally, the rainfall quantities were applied to the NRCS Type II storm distribution. Although this combination of Atlas 14 data with the NRCS Type II storm distribution results in a relatively conservative rainfall pattern, this approach is consistent with the guidelines in *PA Stormwater BMP Manual (2006)*.

SUBWATERSHED AREA

Generally, the subwatershed area for the modeled watersheds was 3-5 mi^2. The drainage areas may be slightly larger or smaller depending on hydrologic characteristics and location of problem areas. Subwatersheds with an area less than one (1) square mile were included in the model if they formed a junction between two larger basins or were tributary to a defined problem area.

Basins with drainage area outside of the scope of this Plan (i.e., the Act 167 designated watershed of the Genesee River) were beyond the scope of study so they were not studied at the same level of detail as portions of the watershed within the scope of this Plan. Generally, they were delineated into areas between 10 and 25 mi^2 and were assumed to have only negligible changes in hydrology due to future land use.

SOILS

Soil properties, specifically infiltration rate and subsurface permeability, are an important factor in runoff estimates. Runoff potential of different soils can vary considerably. Soils are classified into four Hydrologic Soil Groups (A, B, C, and D) according to their minimum infiltration rate (SCS 1986). HSG A refers to soils with relatively high permeability and favorable drainage characteristics; HSG D soils have relatively low permeability and poor drainage characteristics. The runoff potential increases dramatically in order of group A (lowest), B, C, and D (highest). Soil cover data was used in conjunction with land use cover data within GIS to develop composite curve numbers for each subwatershed in the models.

In Section III, Table 3.5 shows the relative percentage of hydrologic soil groups in Potter County. Generally, the runoff potential of soils in the northwestern portion of the county is very high; the location of these soil types corresponds to the location of many of the county's identified problem areas.

LAND USE

Existing land use was derived from the Potter County Planning Commission and are listed within Tables in Section 6. This data was converted to land uses that correspond to NRCS curve number tables (*NRCS*, 1986). The land use categories that were used are listed in *Table A.2*.

LAG TIME

Lag time is the transform routine when using the NRCS Curve Number Runoff Method. Lag can be related to time of concentration using the empirical relation:

$$T_{Lag} = 0.6 * T_C$$

Lag time values for the subwatersheds were based on NRCS Lag Equation and altered as depicted in the tables at the end of this section:

$$T_{Lag} = L^{0.8} \frac{(S+1)^{0.7}}{1900\sqrt{Y}}$$

Where: T_{lag} = Lag time (hours)

L = Hydraulic length of watershed (feet)

Y = Average overland slope of watershed (percent)

S = Maximum retention in watershed as defined by: S = [(1000/CN) – 10]

CN = Curve Number (as defined by the NRCS Rainfall-Runoff Method)

For comparison purposes, a lag time was also calculated for each subwatershed using the TR-55 segmental method. Given the rural landscape of Potter County, the best estimate for time of concentration calculation was provided by the NRCS lag equation.

REACH LENGTHS, SLOPES, AND CROSS SECTION DIMENSIONS

Reach lengths and slopes were determined within GIS. Channel baseflow widths and depths for each river reach were estimated based on drainage area and percent carbonate using the methodology outlined in *Development of Regional Curves Relating Bankfull-Channel Geometry and Discharge to Drainage Area for Streams in Pennsylvania and Selected Areas of Maryland*

(USGS, 2005). Dimensions for the overbank area were visually determined from FEMA floodplains or visual inspection of topographic data. *Figure A.3* shows the dimensions as they are approximated.

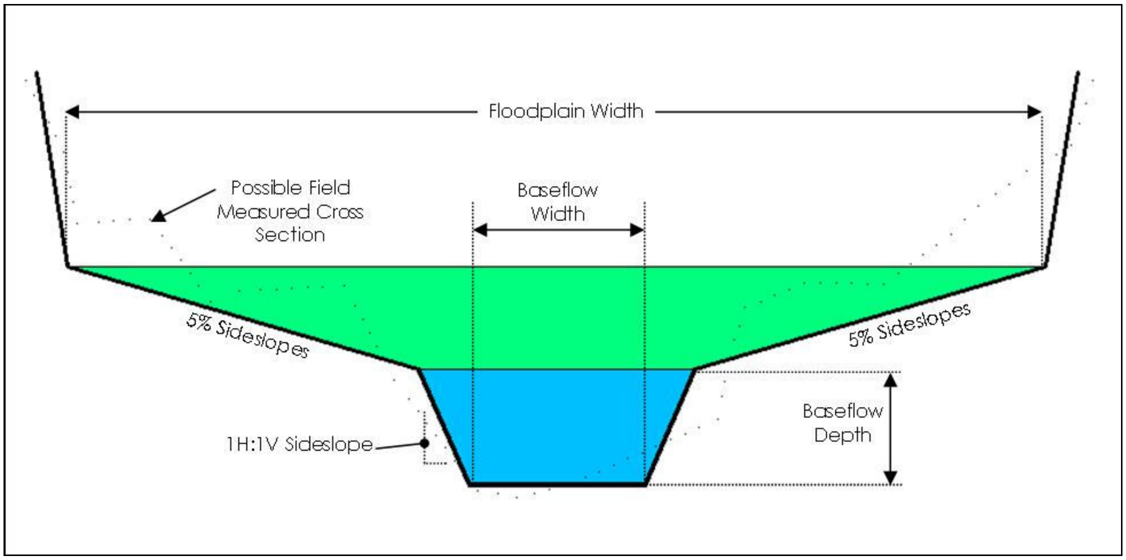


Figure A.3. Cross Sections Used for Reaches in HEC-HMS Model

The reaches were modeled using the Muskingum-Cunge routing procedure. This procedure is based on the continuity equation and the diffusion form of the momentum equation. Manning's Roughness Coefficient *n* values were assumed to be 0.055 in channel; overbank channel values were assumed to be 0.08. When necessary for calibration, Manning's *n* values and the overbank sideslopes were altered so that realistic discharge values could be obtained. The data used for each specific reach is available within the HEC-HMS Models.

INFILTRATION AND HYDROLOGIC LOSS ESTIMATES

Infiltration and all other hydrologic loss estimates (e.g., evapotranspiration, percolation, depression storage, etc.) taken into account within the HEC-HMS model was consistent with the recharge volume criteria contained in Control Guidance 1 and 2 (CG-1 and CG-2). These losses were modeled in existing conditions as the standard initial abstraction in the NRCS Curve Number Runoff method (i.e., $I_a = 0.2S$). CG1 was simulated by modifying the standard initial abstraction using the following procedure.

The runoff volume is computed by HEC-HMS using the following equation:

$$Q_{volume} = \frac{(P - I_a)^2}{(P - I_a) + S}$$

Where P = rainfall for a specific storm event (in),
I_a = initial abstraction (in), and
S = maximum retention (in).

S is defined by the following equation which relates runoff volume to curve number:

$$S = \frac{1000}{CN} - 10$$

The standard initial abstraction I_a used in Pennsylvania is typically *0.2S*. HEC-HMS calculates this automatically if no value is entered by the user. This was the approach used for the existing and future conditions modeling scenarios.

In future conditions with implementation of CG-1, the following equation is applicable. The goal of CG-1 is to ensure there is no discharge volume increase for the 2-year storm event, so

$$Q_{CG1} = Q_{Existing} = \frac{(P - I_a)^2}{(P - I_a) + S_{Proposed}}$$

Where P = rainfall for a specific storm event(in),
I_a = initial abstraction (in), and
$S_{Proposed}$ = maximum retention in proposed conditions as a function of the proposed conditions curve number (in).

Assuming $I_a = 0.2S$ as the Initial abstraction is no longer applicable with CG-1 since BMPs are to be installed to control or remove the increase in runoff volume for the 2-year storm. Using the HEC-HMS modeling output for $Q_{Existing}$, the initial abstraction for CG-1 may be calculated using the following equation:

$$I_a = P_{2-year} - \frac{1}{2}\left(Q_{Existing} \pm \sqrt{Q_{Existing}^2 + 4Q_{Existing}S_{Proposed}}\right) \text{ for the 2-year event}$$

Thus, the volume control required by CG-1 is implicitly modeled by overriding the HEC-HMS default for initial abstraction with the above value. The qualitative effect of this will be to eliminate the increase in runoff volume for the 2-year storm and to reduce the increase in runoff volume of the more extreme events. Increases in the peak flow values are reduced for all storms, but not eliminated, since the time of concentrations for proposed condition are decreased. Figure A.4 shows the effects of implementing a CG-1 policy on an example watershed. In the first figure representing a 2-year storm event, the hydrograph volumes are exactly the same and the peaks are similar. In the second figure representing a 100-year storm event, the hydrograph volumes are not the same since only the 2-year volume is abstracted; consequently there is still a substantial increase in peak flows, although the CG-1 implementation does reduce the peak flow.

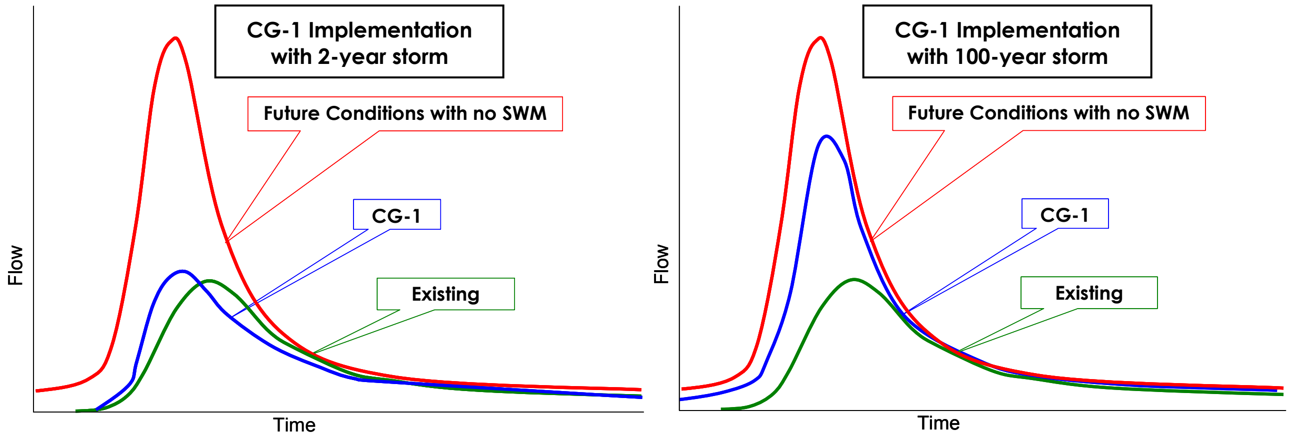


Figure A.4. Typical On-Site Runoff Control Strategy

In the case of this particular sample, release rates might be necessary to prevent increases in peak flow. In situations where there is only a small increase in impervious coverage, however, CG-1 may reduce the proposed conditions peak flow to existing conditions levels without the use of release rates.

For the 2-year event, modeling CG-1 with the above equations results in an increased approximation in initial abstraction represented by D:

$$D = I_a^{CG-1} - 0.2S$$

For every event of greater magnitude (e.g., 10, 25, 50, and 100-year events), the initial abstraction is calculated using the sum of the traditional method and the increase in initial abstraction for the 2-year event.

$I_a = 0.2S + D$ for all events greater than the 2-year event.

MODEL CALIBRATION

Three parameters were modified to develop a calibrated hydrologic model: the curve number, the time of concentration, and the Manning's coefficient used in the Muskingum-Cunge routing method.

The antecedent runoff condition was altered for each storm event so that each subbasin and calibration point was within an acceptable range of a target flow. The equation used to modify antecedent runoff condition (Maryland Hydrology Panel, 2006):

For ARC≤2:

$$CN_x = \frac{[10 + 5.8(x-2)]CN_2}{10 + 0.058(x-2)CN_2}$$

For ARC>2:

$$CN_x = \frac{[10 + 13(x - 2)]CN_2}{10 + 0.013(x - 2)CN_2}$$

Thus a unique ARC and resulting curve number was calculated for each subbasin for each storm event. The same ARC was applied in both existing and proposed conditions. The calibrated and future condition curve numbers for the two watersheds are presented in the Tables at the end of this appendix.

Additionally, lag times were calculated using both TR-55 and the NRCS lag equation. The initial model runs used the results from the NRCS lag equation. A factor between 0 and 2 was applied to the initial value to obtain a calibrated time of concentration value. The same time of concentration was applied to all existing condition storms. The future land use time of concentration was calculated using the NRCS lag equation with future land use curve numbers and it was subsequently adjusted by the same factor used in existing conditions.

Finally the Manning's n value for channels and overbank areas was modified to obtain realistic flow values. The respective ranges for the channel and overbank areas were 0.02-0.07 and 0.03-0.2.

The accuracy of the model remains unknown unless it is calibrated to another source of runoff information. Possible sources of information include stream gage data, high water marks (where detailed survey is available to facilitate hydraulic analysis), and other hydrologic models. The most desirable source of calibration information is stream gage data as this provides an actual measure of the runoff response of the watershed during real rain events.

USGS Gage 03010655 Oswayo Creek at Shinglehouse, PA

There are four USGS stream gages with adequate record located in Potter County. Two of the gages were associated with Oswayo Creek and Genesee River. The following table lists these gages and their respective statistics.

USGS Stream Gage No.	Site Name	Drainage Area mi²	Number of Gage Years at Gage	Used in HEC-HMS Model
01543700	First Fork Sinnemahoning Creek at Wharton, PA	182.0	26	Not Used
01544450	Germania Branch at Germania, PA	2.4	11	Not Used
03010655	Oswayo Creek at Shinglehouse, PA	98.7	34	Limited Use
04221000	Genesee River at Wellsville, NY	288	41	Limited Use

Table A.6. USGS Stream Gages Associated with Oswayo Creek and Genesee River

Available hydrologic data for the Oswayo Creek watershed is limited and of questionable value to our particular approach in this Plan. USGS Stream Gage 03010655 within Shinglehouse Borough has 34 years of measured data. Between 1975 and 2008, annual peak flow values range between 923 cfs and 4,660 cfs (a factor of 5). The validity of this data, however, was questioned in the 1991 FIS for the Borough (FEMA, 1991). The FIS stated that the gage was not in place during large flood events in recent years. The USGS has no record of this gage's operation, nor can it confirm the assertion in the 1991 FIS, and it was not able to provide any additional information regarding irregular data collection techniques at the gage site (USGS, 2009). Since large flood events are the focus of this Plan and the Bulletin 17B analysis used for calibration, this gage was only qualitatively considered in this Plan.

When no stream gage data is available, the next most desirable source of data for purposes of comparison is other hydrologic studies prepared by local, state, or federal agencies. FEMA Flood Insurance Studies (FIS) often provide discharge estimates at specific locations within FEMA floodplains. The estimates provided in FEMA FISs are valid sources for comparison but should be carefully considered when used for calibration since they are sometimes dependent on outdated methodology, or focus exclusively on the 100-year event for flood insurance purposes.

The third available source of information that may be used for calibration is regression equation estimates. The regression equations were developed on the basis of peak flow data collected at numerous stream gages throughout Pennsylvania. This procedure is the most up-to-date method and takes into account watershed average elevation, carbonate (limestone) area, and minor surface water storage features such as small ponds and wetlands. The methodology for developing regression equation estimates within Pennsylvania is outlined in USGS Scientific Investigations Report 2008-5102 (USGS, 2008). Mean Elevation, Percent Carbonate Rock, and Percent Storage, the applicable parameters within Potter County, were calculated using GIS from layers supplied from USGS Digital Elevation Model (DEM) data, Environmental Resources Research Institute (1996), and USGS (2008).

The target flow rates were determined from one of these three sources. The HEC-HMS models were then calibrated to the target flow rates at the overall watershed level, at subwatersheds where significant hydrologic features were identified (e.g., confluences, dams, USGS Gages), and at each individual subbasin. This approach was used so that a flow value anywhere in the model would compare favorably to the best available data source. The parameters of calibration for the entire overall watershed were the antecedent runoff condition, lag time, and reach routing coefficients. Detailed calibration results are provided in the form of tables at the end of this section.

The following figures (*Figures A.5-A.17*) show the overall watershed calibration results for Oswayo Creek and Genesee River. As can be shown, the calibration results are in general agreement with the range of values for other hydrologic studies with the exception of USGS Stream Gage 03010655 Oswayo Creek at Shinglehouse, PA.

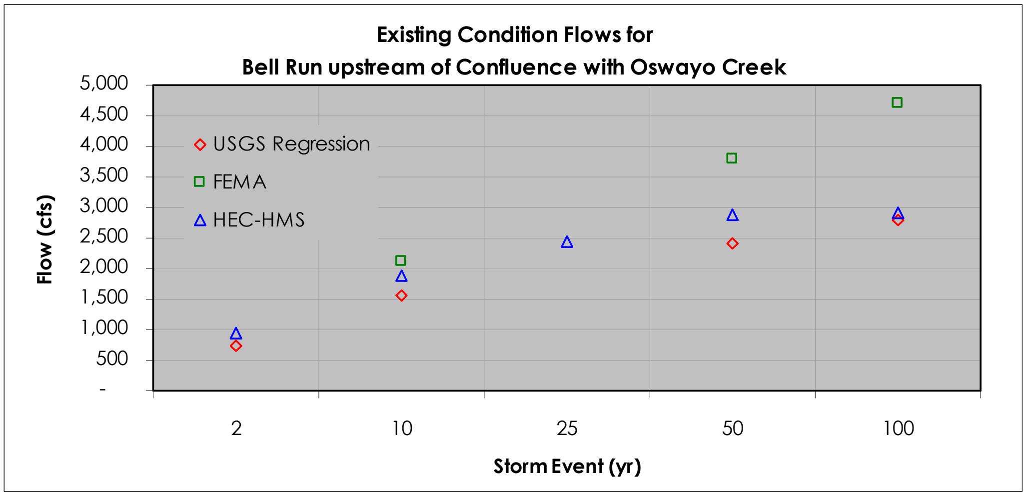


Figure A.5.

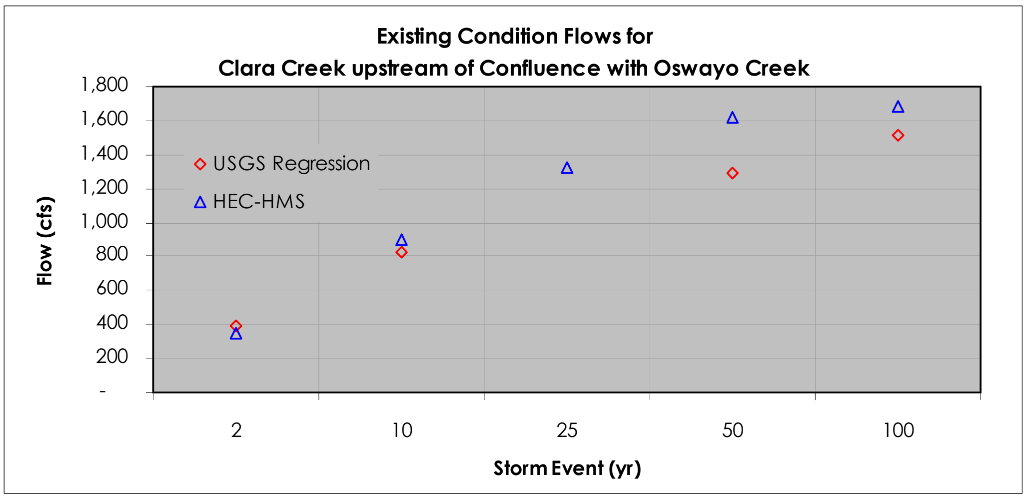
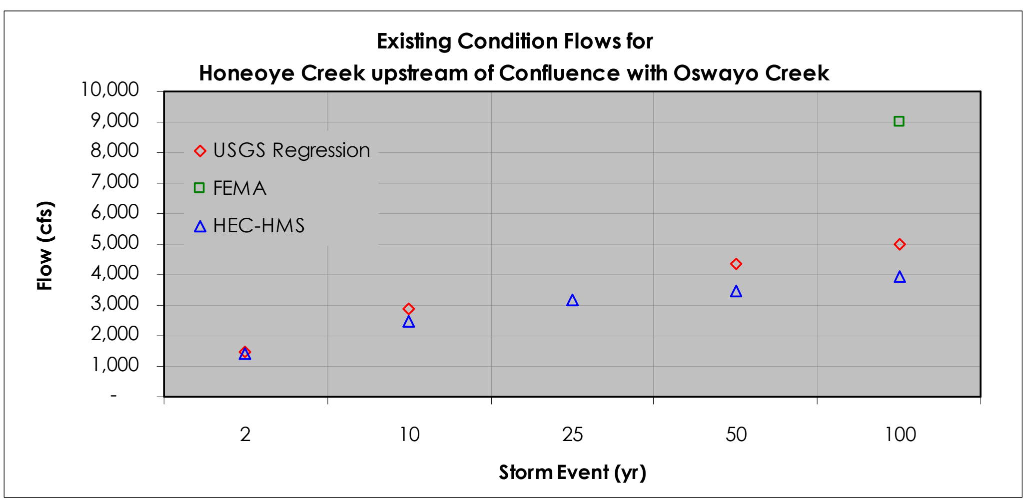


Figure A.6.



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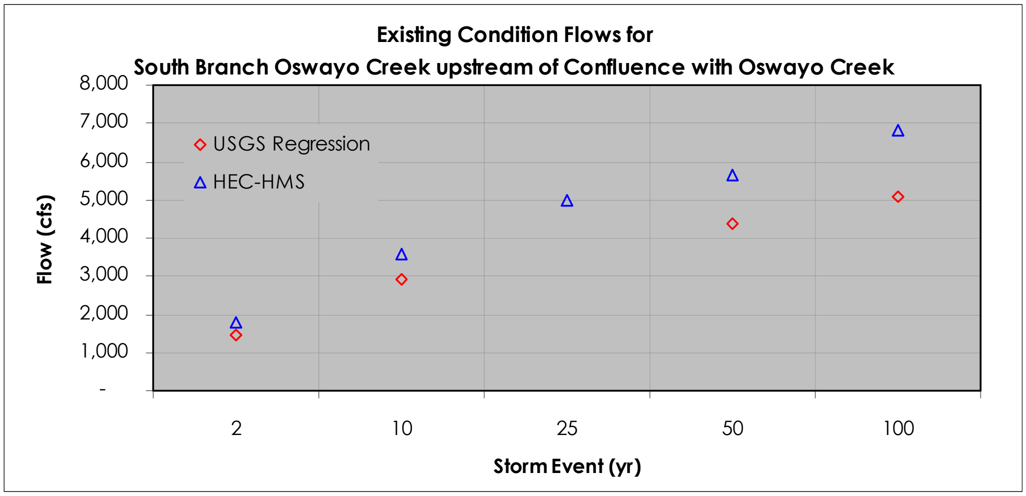

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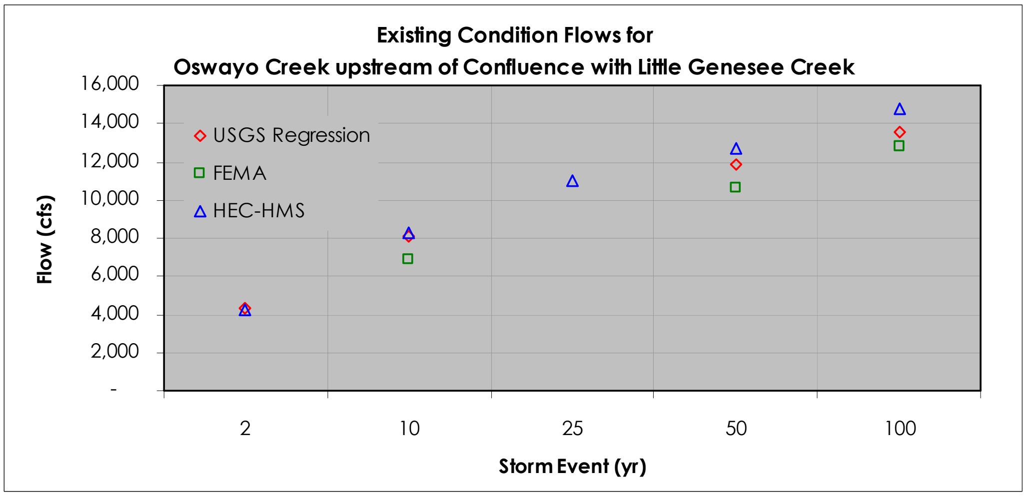


Figure A.9.

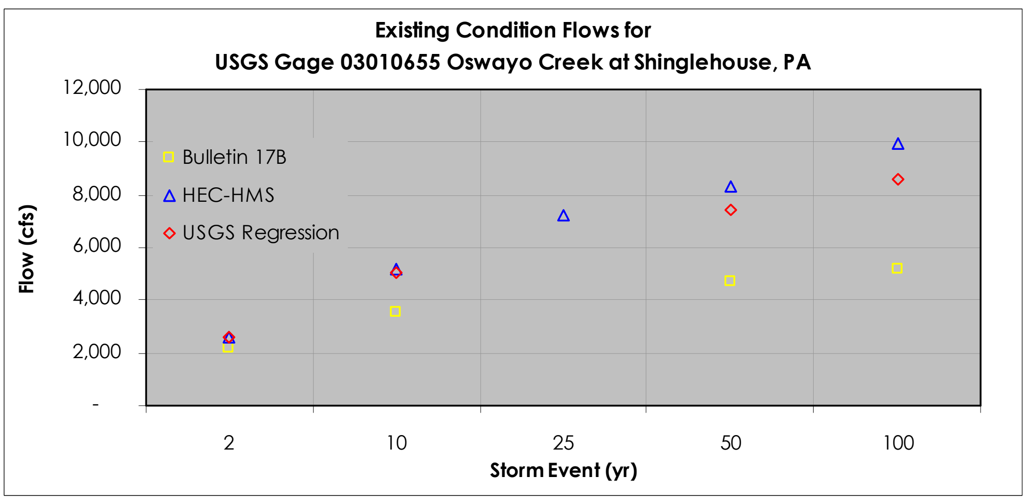
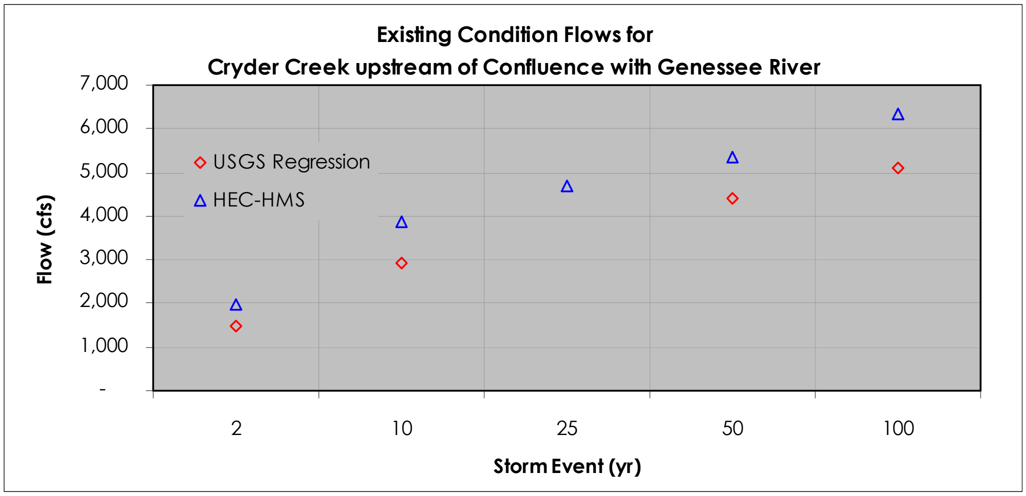


Figure A.10.



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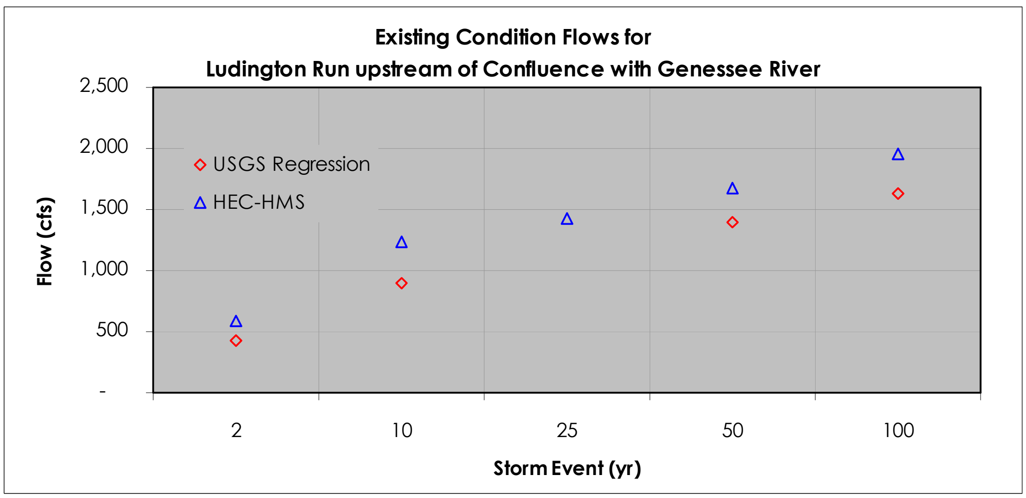

Figure A.12.

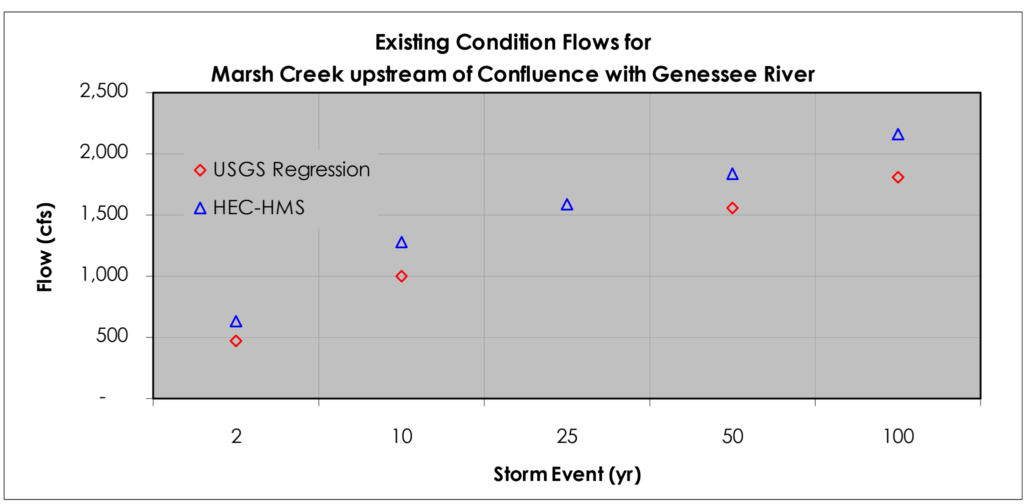

Figure A.13.

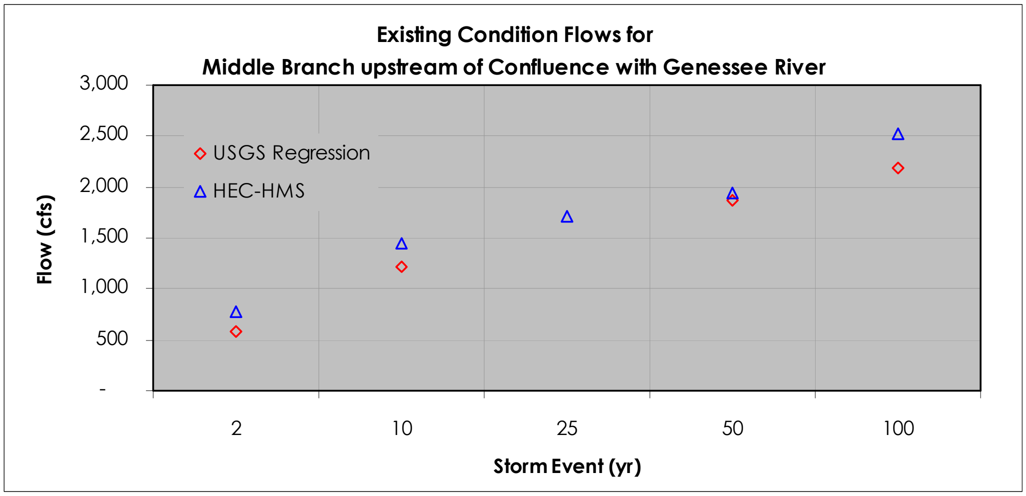

Figure A.14.

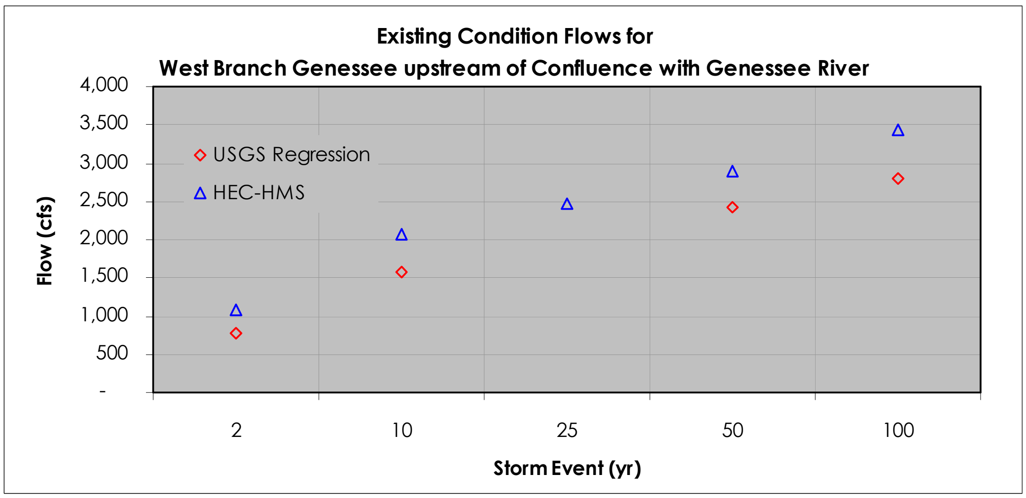

Figure A.15.

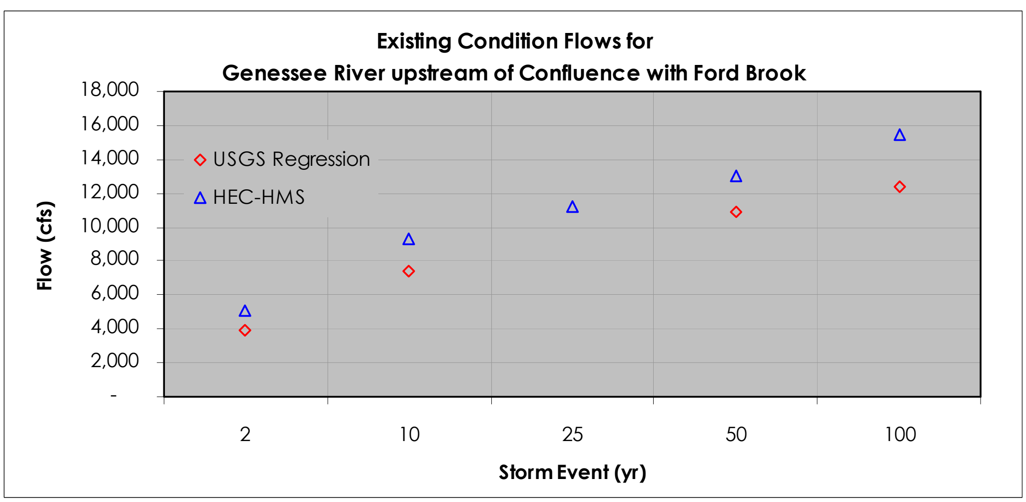

Figure A.16.

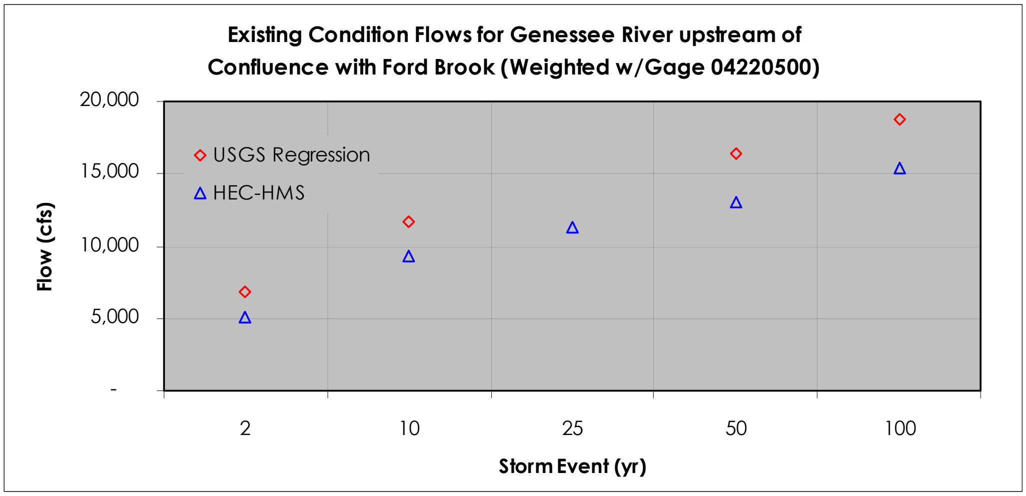

Figure A.17.

MODELING RESULTS

Once the existing conditions model was calibrated and the existing conditions peak flows were established, additional models were developed to assist in determining appropriate stormwater management controls for the watersheds. Based on a comparison of existing and future land use, most subbasins will experience varying degrees of development through the full build-out future condition.

The following simulations were performed with HEC-HMS (2, 10, 25, 50 and 100-year) for Oswayo Creek and Genesee River:

Existing Conditions (Ex)

An existing conditions model was developed and analyzed using the using the calibration procedures described above. Results from the existing conditions model reflect the estimated land uses from 2010. The existing condition flows are provided in the form of tables at the end of this section.

Future Conditions with No Stormwater Controls (F-1)

A future conditions model was developed and analyzed using the projected future land use coverage for the year 2020 provided by the Potter County Planning Department. The revised land use resulted in an increased curve number and a decreased time of concentration for several subbasins. It was assumed that there was no required detention or any other stormwater controls in this simulation.

Future Conditions with Design Storm Method and Release Rates (CG-1R)

A future conditions model with Stormwater Controls was developed by modifying the future conditions model to include the effects of peak rate controls and the volume removal requirements of the Design Storm Method.

The effects of peak rate controls, through detention of post development flows, was estimated by routing the post development flow for each subbasin through a simulated reservoir. The reservoirs were designed so that they could release no more than the pre-development flow estimate. This approach was assumed to simulate the additive effect of all of the individual detention facilities within a sub-basin. The volume removal requirements of the Design Storm Method were simulated using modified initial abstraction values as described above and illustrated in the form of tables at the end of this section.

The approach in this Act 167 Plan was to 1) estimate the effects of detention of post development flows and 2) apply release rates to subwatershed wherever there is a significant increases in peak flow at the points of interest. The results for each watershed are presented below; detailed results of the modeling are provided at the end of this section.

OSWAYO CREEK

For the Oswayo Creek Watershed, the projected future increases are located mostly near the Towns of Shinglehouse and Oswayo which are geographically centered in the western and southeastern part of the watershed. This development pattern indicates the potential need for peak rate controls more stringent than the traditional 100% release rates. The increases within Oswayo Creek are depicted in *Figure 6.1*.

Storm Event (year)	Effects of Future Condition on Discharges		
	Maximum % Increase in Future Conditions	Average % Increase in Future Conditions[1]	Portion of subbasins with Increase (%)
2	78.3	2.2	19.8
10	61.3	2.0	19.8
25	58.5	1.8	19.8
50	60.8	1.6	19.8
100	57.9	1.6	19.8

Notes: [1] Area weighted averages

Table A.7. Future Condition Flows with No Stormwater Management Controls for Oswayo Creek

Table A.8 shows the reduction in peak flows that would occur if only the Design Storm Method were implemented without any peak rate controls. The flows for the lower magnitude events are substantially reduced compared to future conditions with no stormwater management controls with the implementation of the Design Storm Method. The flows for the higher magnitude events are moderately reduced with implementation of the Design Storm Method, but significant increases still occur.

Storm Event (year)	Effects of CG-1 on Discharges		
	Maximum % Increase with CG1	Average % Increase with CG1[1]	Portion of subbasins with Increase (%)
2	1.5	0.2	21.9
10	16.1	0.7	19.8
25	23.1	0.8	16.7
50	28.3	0.8	16.7
100	31.2	0.9	15.6

Notes: [1]Area weighted averages

Table A.8 Future Subbasin Flows with Design Storm Method Only – No peak control for Lake Oswayo Creek

GENESEE RIVER

For the Genesee River Watershed, the projected future increases occur around the Towns of Genesee and Ulysses which are geographically located in the center and southeastern part of the watershed in Potter County. This development pattern also indicates the potential need for peak rate controls more stringent than the traditional 100% release rates. The increases within the Genesee Creek watershed are depicted in *Figure 6.2*.

Storm Event (year)	Effects of Future Condition on Discharges		
	Maximum % Increase in Future Conditions	Average % Increase in Future Conditions[1]	Portion of subbasins with Increase (%)
2	200.4	2.3	20.7
10	150.9	1.9	20.7
25	156.2	1.9	20.7
50	151.5	1.9	20.7
100	148.3	1.9	20.7

Notes: [1]Area weighted averages

Table A.9. Future Condition Flows with No Stormwater Management Controls for the Genesee River (within Potter County)

Table A.10 shows the reduction in peak flows that would occur if only the Design Storm Method were implemented without any peak rate controls.

Storm Event (year)	Effects of CG1 on Discharges		
	Maximum % Increase with CG1	Average % Increase with CG1[1]	Portion of subbasins with Increase (%)
2	1.0	0.3	20.7
10	29.4	0.5	17.1
25	48.7	0.8	20.7
50	61.6	1.0	19.5
100	72.1	1.1	20.7

Notes: [1] Area weighted averages

Table A.10. Future Subbasin Flows with Design Storm Method Only – No peak control for the Genesee River (within Potter County)

STORMWATER MANAGEMENT DISTRICTS

The regional philosophy used in Act 167 planning introduces a different stormwater management approach than is found in the traditional on-site approach. The difference between the on-site stormwater control philosophy and the Act 167 watershed-level philosophy is the consideration of downstream impacts throughout an individual watershed. The objective of typical on-site design is to control post-development peak flow rates from the site itself; however, a watershed-level design is focused on maintaining existing peak flow rates in the entire drainage basin. The watershed approach requires knowledge of how the site relates to the entire watershed in terms of the timing of peak flows, contribution to peak flows at various downstream locations, and the impact of the additional runoff volume generated by the development of the site. The proposed watershed-level stormwater runoff control philosophy is based on the assumption that runoff volumes will increase with development and the philosophy seeks to manage the increase in volumes such that peak rates of flow throughout the watershed are not increased. The controls implemented in this Plan are aimed at minimizing the increase in runoff volumes and their impacts, especially for the 2-year storm event.

The basic goal of both on-site and watershed-level philosophies is the same, i.e. no increase in the peak rate of stream flow. The end products, however, can be very different as illustrated in the following simplified example.

Presented in *Figure A.18* is a typical on-site runoff control strategy for dealing with the increase in the peak rate of runoff with development. The Existing Condition curve represents the pre-development runoff hydrograph. The Developed Condition hydrograph illustrates three important changes in the site runoff response with development:

1. A higher peak rate,

2. A faster occurring peak (shorter time for the peak rate to occur), and

3. An increase in total runoff volume.

The "Controlled" Developed Condition hydrograph is based on limiting the post-development runoff peak rate to the pre-development level through use of detention facilities; but the volume is still increased. The impact of "squashing" the post-development runoff to the pre-development peak without reducing the volume is that the peak rate occurs over a much longer period of time. The instantaneous pre-development peak has become an extended peak (approximately two (2) hours long in this example) under the "Controlled" Developed Condition.

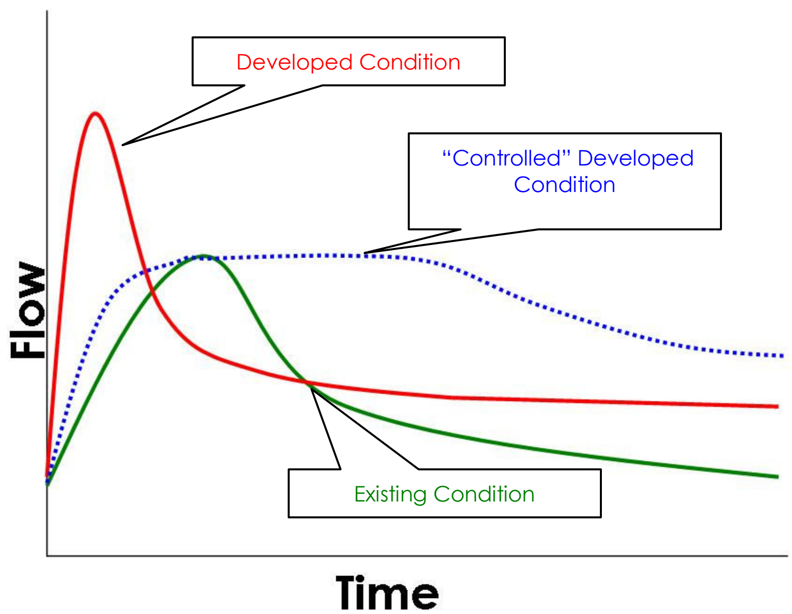

Figure A.18. Typical On-Site Runoff Control Strategy

Considering the outflow from the site only, the maintenance of the pre-development peak rate of runoff is an effective management approach. However, *Figures A.19* and *A.20* illustrate the potential detrimental impact of this approach. *Figure A.19* represents the existing hydrograph at the point of confluence of Watershed A and Watershed B. The timing relationship of the watersheds is that Watershed A peaks more quickly (at time T_{pA}) than the Total Hydrograph, while Watershed B peaks later (at time T_{pB}), than the Total Hydrograph, resulting in a combined time to peak approximately in the middle (at time T_p). Watershed A is an area of significant development pressure, and all new development proposals are met with the on-site runoff control philosophy as depicted in *Figure A.18*. The eventual end product of the Watershed A development under the "Controlled" Development Condition is an extended peak rate of runoff as shown in *Figure A.20*. The extended Watershed A peak occurs long enough so that it coincides with the peak of Watershed B. Since the Total Hydrograph at the confluence is the summation of Watershed A and Watershed B, the Total Hydrograph peak is increased under these conditions to the "Controlled" Total Hydrograph. The conclusion from the example is that simply controlling peak rates of runoff on-site does not guarantee an effective watershed level of control because of the increase in total runoff volume. The net result is that downstream peaks can increase and extend for longer durations.

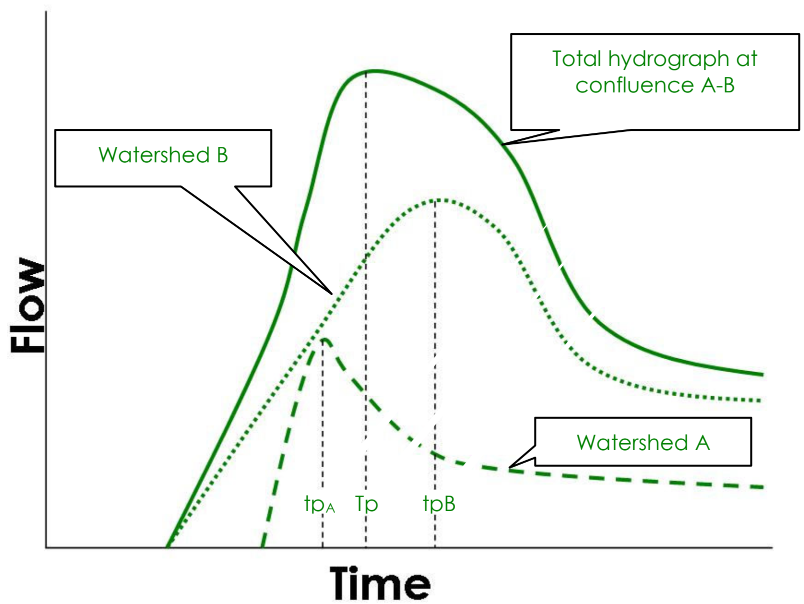

Figure A.19. Existing Hydrograph (Pre-Development)

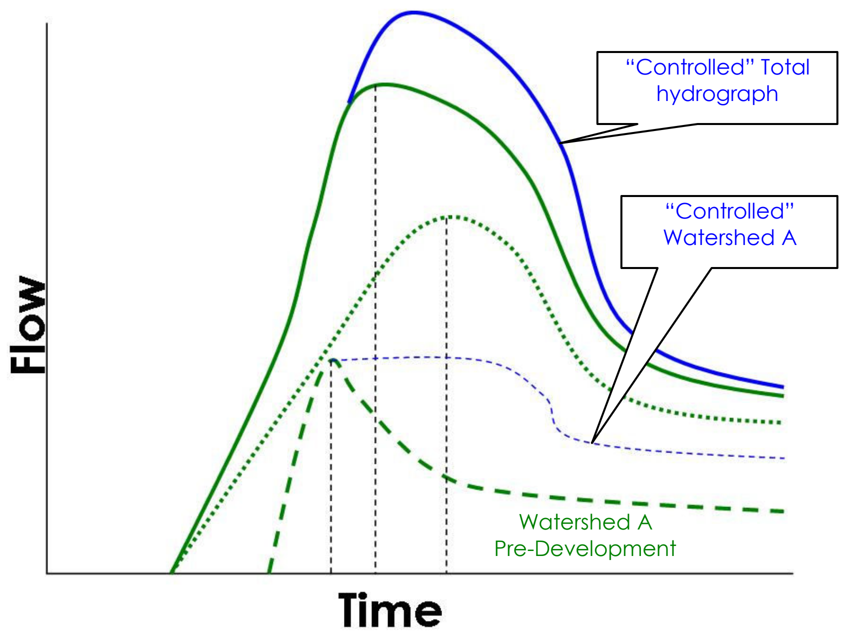

Figure A.20. Controlled Runoff Condition (Post-Development)

RELEASE RATE CONCEPT

The previous example indicated that, in certain circumstances, it is not enough to control post-development runoff peaks to pre-development levels if the overall goal is no increase in peak runoff at any point in the watershed. The reasons for this potential increase are how the various parts of the watershed interact, in time, with one another and the increased rate and volume of runoff associated with development and increases in impervious surfaces. The critical runoff criteria for a given site or watershed area is not necessarily its own pre-development peak rate of runoff but rather the pre-development contribution of the site or watershed area to the peak flow at a given point of interest.

To account for increases of volume and peak flow resulting from the combination of these post-development hydrographs, stormwater management districts have been assigned to various areas within the county boundary that have more restrictive release rates than the conventional 100% release rate. As shown in Plate 10, some areas within specific watersheds have reduced release rates where CG-1 may be difficult to completely implement.

The specification of a 100% release rate as a performance standard would represent the conventional approach to runoff control philosophy, namely controlling the post-development peak runoff to pre-development levels. This is a well-established and technically feasible control that is effective at-site and, where appropriate, would be an effective watershed-level control.

It is important to acknowledge that there are several problems with the release rate concept. One of the problems is that some areas can reach unreasonably low release rates. This can be seen in the release rate equation, which dictates that sub-watersheds which peak farther away from the entire watershed will have a lower release rate. Indeed, sub-watersheds whose runoff drains almost completely before or after the watershed peak will approach a release rate of zero (because the numerator approaches zero).

Another problem is that release rates are highly dependent on, and sensitive to, the timing of hydrographs. Since natural storms follow a different timing than design storms, it is still possible that watershed wide controls designed with release rates only, will encounter increased runoff problems. This is because the runoff rates are still much higher in the developed condition, and increased volumes over an extended time can combine to increase peak flow rates. Similar to the traditional on-site detention pond, release rates are purely a peak "rate" type of control.

Patterns of development may also determine how effective designs are that use only release rates, or any control based on timing. This is because rates based on timing assume a certain development and rainfall patterns, and the model uses uniform parameters across a sub-watershed. In reality, the actual development and rainfall patterns can be highly variable across a sub-watershed and can be quite different than the "Future Full Build Out" land use scenario used in the planning study. This uncertainty can affect any type of control, but controls based on timing alone are especially sensitive to these parameters. Some controls, such as volume controls, are less sensitive since they remove a certain amount of runoff from the storm event wherever development occurs. In a sense, volume controls tend to more closely simulate what occurs in a natural system.

Combining volume controls with peak rate controls, as proposed in this plan, will be more effective than having only peak rate controls. Volume controls have several advantages such as:

1. Increased runoff volume may infiltrate and provide recharge to existing groundwater supplies. This may not happen with rate controls since all of the runoff excess is discharged in a relatively short time frame.

2. Volume controls tend to mimic natural systems (i.e., excess runoff volume is infiltrated) and thus are more effective in controlling natural storms since they are not highly sensitive to timing issues.

3. Volume controls often have enhanced water quality benefits.

4. The Design Storm Method and The Simplified Method as implemented in this Plan, provide the benefits described above.

SUMMARY MODEL OUTPUT

Hydrologic Parameters for Oswayo Creek HEC-HMS Model

Hydrologic Results for Oswayo Creek HEC-HMS Model

Hydrologic Parameters for Genesee River HEC-HMS Model

Hydrologic Results for Genesee River HEC-HMS Model

Calibration Results for Detailed HEC-HMS Models with 2010 Land Use

Hydrologic Parameters for Oswayo Creek HEC-HMS Model

Subwatershed Name	Subbasin	Drainage Area (mi^2)	Existing Conditions (2010)		Future Conditions (2020)	
			CN	Lag (min)	CN	Lag (min)
Bell Run	W310	3.00	60.4	68.7	60.4	68.7
	W710	3.21	61.8	63.4	61.8	63.4
	W720	4.72	59.7	71.1	59.7	71.1
	W730	4.40	60.8	94.0	60.8	94.0
Bradley Run	W110	1.02	58.0	44.0	58.0	44.0
	W120	0.48	63.5	28.2	63.5	28.2
	W130	1.05	63.3	43.6	63.3	43.6
	W140	1.27	63.5	44.9	63.5	44.9
	W150	0.55	61.6	36.4	61.6	36.4
	W160	0.08	62.2	15.4	62.2	15.4
Butter Creek	W250	2.80	70.2	59.8	70.2	59.8
	W260	1.71	65.9	50.6	65.9	50.6
Canada Run	W530	1.36	62.9	38.4	62.9	38.4
	W540	1.82	64.6	55.6	64.7	55.5
Clara Creek	W010	1.03	63.5	34.8	63.5	38.7
	W020	0.96	61.9	35.4	61.9	35.4
	W420	0.96	63.6	38.7	63.6	38.7
	W430	0.06	67.7	12.1	67.7	12.1
	W440	0.43	65.8	25.0	65.8	25.0
	W450	1.74	64.3	53.2	64.3	53.2
Cow Run	W570	2.35	65.0	62.1	65.0	62.1
Elevenmile Creek	W170	2.28	71.0	62.9	71.0	62.9
	W180	1.79	70.7	49.0	70.7	49.0
	W470	1.63	72.4	50.7	72.4	50.7
	W480	3.12	69.3	58.2	69.3	58.2
	W490	2.47	68.4	73.8	68.4	73.8
	W500	1.91	67.0	39.9	67.0	39.9
	W510	0.77	65.9	33.1	65.8	33.1
	W520	1.44	68.9	43.1	68.9	43.1
Hemlock Hollow Run	W040	0.92	66.1	50.3	66.1	50.3
	W050	0.89	69.9	41.1	69.9	41.1
	W060	0.44	67.3	30.5	67.3	30.5
	W070	3.62	67.8	82.8	67.8	82.8
	W330	2.37	69.5	54.2	69.5	54.2
	W340	2.28	66.8	46.7	66.8	46.7
	W350	0.08	64.0	19.6	64.3	19.4
Honeoye Creek	W190	1.05	69.8	38.6	69.8	38.6
	W200	2.60	70.3	62.4	70.3	62.4
	W210	0.53	67.4	28.4	67.4	28.4
	W220	1.29	69.4	51.4	69.4	51.4
	W230	6.78	66.7	98.7	66.7	94.0
	W240	8.54	66.6	103.9	66.6	98.9
	W270	1.63	67.0	47.4	67.4	46.9
	W580	6.98	68.5	95.0	68.5	86.4

Hydrologic Parameters for Oswayo Creek HEC-HMS Model

Subwatershed Name	Subbasin	Drainage Area (mi^2)	Existing Conditions (2010)		Future Conditions (2020)	
			CN	Lag (min)	CN	Lag (min)
Honeoye Creek	W590	1.25	66.7	47.2	66.7	45.0
	W600	0.00	68.5	13.3	68.5	13.3
	W610	5.95	67.7	99.9	67.7	99.9
	W620	1.59	67.2	73.2	67.3	60.9
	W630	0.00	67.0	8.8	69.9	8.2
	W640	1.72	64.8	48.7	64.9	48.6
	W650	1.93	68.8	46.2	68.8	46.2
	W660	2.46	68.8	75.3	71.6	58.2
Horse Run	W680	7.53	67.6	114.8	67.7	114.5
Janders Run	W670	1.37	59.1	53.2	59.8	52.3
Oswayo Creek	W320	1.89	68.7	55.6	68.7	55.6
	W360	1.71	61.3	50.6	62.5	49.1
	W370	2.99	60.1	59.3	60.5	65.1
	W460	1.08	61.2	38.4	61.2	38.4
	W550	1.01	61.9	43.1	61.9	43.1
	W740	1.02	62.2	56.4	62.2	56.4
	W750	0.81	70.9	34.9	70.9	34.9
	W760	1.98	68.2	47.3	68.2	47.3
	W770	2.43	65.1	47.3	65.1	47.3
	W780	1.32	65.3	43.3	65.3	43.3
	W790	0.22	69.1	23.5	76.0	19.4
	W800	0.00	76.4	11.6	90.9	7.1
	W810	3.94	61.3	62.9	61.9	65.1
	W820	2.01	61.6	53.5	61.9	56.0
	W830	2.58	60.7	63.3	60.7	66.7
	W840	0.03	55.8	41.7	55.8	41.7
	W850	0.57	64.8	43.0	64.8	43.0
	W860	0.81	63.6	43.3	64.9	41.8
	W870	1.91	69.5	68.1	69.6	67.9
	W880	0.08	68.0	13.5	68.0	13.5
	W890	2.68	69.9	76.5	69.9	76.5
	W900	2.18	66.4	70.9	67.2	69.4
	W910	0.80	65.6	94.3	70.6	82.6
	W920	0.17	66.7	45.1	72.7	38.4
	W930	0.63	59.9	45.9	60.1	45.7
	W940	0.28	67.1	52.9	67.1	52.9
	W950	0.00	65.4	48.1	65.4	48.1
	W960	1.19	63.4	76.6	63.4	76.6
Plank Creek	W280	2.06	70.1	63.3	70.1	63.3
Shaytown Branch	W290	1.23	60.6	40.8	60.6	40.8
	W300	0.35	60.2	36.7	60.2	36.7
	W690	1.69	58.9	61.6	58.9	61.6
	W700	1.90	62.6	57.3	62.6	57.3
South Branch Oswayo Creek	W030	2.48	66.0	65.5	66.0	65.5

Hydrologic Parameters for Oswayo Creek HEC-HMS Model

Subwatershed Name	Subbasin	Drainage Area (mi^2)	Existing Conditions (2010)		Future Conditions (2020)	
			CN	Lag (min)	CN	Lag (min)
South Branch Oswayo Creek	W380	3.84	65.4	88.0	65.4	88.0
	W390	3.28	62.8	67.0	63.0	66.7
	W400	3.14	61.2	71.0	61.4	74.4
	W410	1.13	61.3	49.2	61.8	48.7
Whitney Creek	W080	3.11	59.2	60.8	59.2	67.6
	W090	3.57	58.4	95.2	58.4	95.2
	W100	0.14	59.8	15.6	59.8	19.5
Wildcat Creek	W560	3.56	70.3	75.0	70.3	75.0

Hydrologic Parameters for Oswayo Creek HEC-HMS Model

Subwatershed Name	Subbasin	Existing CN (ARC=2)	Calibrated Existing Conditions (Year 2010) Curve Numbers				
			2-Year	10-Year	25-Year	50-Year	100-Year
Bell Run	W310	60.4	67.1	62.2	58.3	54.8	49.2
	W710	61.8	67.1	61.8	58.5	54.6	48.5
	W720	59.7	66.7	61.8	57.5	53.8	47.5
	W730	60.8	68.5	64.0	59.2	55.7	52.0
Bradley Run	W110	58.0	63.5	61.0	56.6	53.3	48.1
	W120	63.5	62.4	57.6	55.3	52.7	46.9
	W130	63.3	63.9	59.1	56.1	52.5	48.1
	W140	63.5	64.1	59.3	56.3	52.7	48.3
	W150	61.6	60.5	55.6	53.3	50.7	48.1
	W160	62.2	61.1	56.2	53.9	51.3	48.2
Butter Creek	W250	70.2	67.7	59.7	56.2	52.1	49.8
	W260	65.9	66.2	60.8	57.7	53.9	50.1
Canada Run	W530	62.9	64.9	59.5	56.1	52.0	48.1
	W540	64.6	64.1	60.1	58.0	55.6	50.5
Clara Creek	W010	63.5	62.8	58.1	54.6	50.4	47.2
	W020	61.9	60.7	55.9	53.5	51.0	45.2
	W420	63.6	62.5	57.7	55.4	52.8	47.0
	W430	67.7	66.7	60.2	56.7	52.4	51.5
	W440	65.8	64.7	58.1	54.5	50.2	44.7
	W450	64.3	64.9	60.1	57.1	53.5	47.7
Cow Run	W570	65.0	64.5	60.6	58.0	55.0	50.7
Elevenmile Creek	W170	71.0	65.1	61.2	57.9	53.9	51.3
	W180	70.7	64.3	60.3	57.2	53.6	51.0
	W470	72.4	64.3	60.0	56.8	53.1	52.8
	W480	69.3	64.1	58.2	55.3	51.9	51.6
	W490	68.4	66.0	61.0	57.5	53.2	48.0
	W500	67.0	63.4	58.0	55.4	52.4	47.7
	W510	65.9	64.8	60.0	57.8	55.3	49.4
	W520	68.9	64.4	58.7	55.9	52.6	49.9
Hemlock Hollow Run	W040	66.1	65.0	60.3	57.3	53.7	49.7
	W050	69.9	66.1	58.5	56.8	54.9	54.1
	W060	67.3	63.3	59.8	56.2	51.9	51.1
	W070	67.8	65.8	61.8	58.0	53.3	48.9
	W330	69.5	64.8	58.9	55.1	50.6	48.9
	W340	66.8	63.9	57.9	54.5	50.5	47.5
	W350	64.0	62.9	58.1	55.8	53.2	47.4
Honeoye Creek	W190	69.8	67.0	58.8	55.9	52.5	49.3
	W200	70.3	67.9	60.0	57.2	54.0	49.9
	W210	67.4	70.5	65.6	58.9	54.6	46.5
	W220	69.4	67.9	61.3	57.9	53.8	48.8
	W230	66.7	69.3	63.1	59.2	54.3	49.5
	W240	66.6	69.5	63.2	59.3	54.5	49.7
	W270	67.0	67.6	60.4	57.0	52.9	49.6
	W580	68.5	67.1	61.6	57.5	52.4	49.4

Hydrologic Parameters for Oswayo Creek HEC-HMS Model

Subwatershed Name	Subbasin	Existing CN (ARC=2)	Calibrated Existing Conditions (Year 2010) Curve Numbers				
			2-Year	10-Year	25-Year	50-Year	100-Year
Honeoye Creek	W590	66.7	64.4	58.3	52.9	48.6	45.7
	W600	68.5	69.1	66.8	63.9	60.4	52.5
	W610	67.7	66.5	62.7	57.1	52.4	48.5
	W620	67.2	68.9	64.6	61.0	56.6	54.2
	W630	67.0	66.0	62.2	60.5	58.7	50.8
	W640	64.8	64.7	61.0	58.1	54.6	50.0
	W650	68.8	66.0	61.9	58.2	53.6	49.7
	W660	68.8	66.7	62.7	60.4	57.9	55.2
Horse Run	W680	67.6	70.7	67.2	60.3	56.1	52.3
Janders Run	W670	59.1	65.9	62.3	58.2	55.6	48.9
Oswayo Creek	W320	68.7	65.2	59.9	56.1	51.5	49.0
	W360	61.3	65.2	59.0	55.5	51.3	48.0
	W370	60.1	64.3	58.6	54.9	50.4	47.9
	W460	61.2	65.1	58.8	56.0	52.8	47.9
	W550	61.9	64.2	60.2	58.3	56.2	50.1
	W740	62.2	69.2	63.1	60.4	57.9	52.8
	W750	70.9	69.9	61.8	59.0	55.6	55.3
	W760	68.2	64.2	58.7	55.4	51.5	49.0
	W770	65.1	63.8	59.0	55.2	50.5	45.9
	W780	65.3	64.1	59.3	55.5	50.8	47.7
	W790	69.1	68.1	59.8	57.1	54.0	53.1
	W800	76.4	75.5	70.1	67.9	65.4	64.7
	W810	61.3	64.4	58.5	55.1	51.2	47.7
	W820	61.6	64.5	58.7	55.3	51.4	48.1
	W830	60.7	64.3	58.2	55.4	52.3	46.8
	W840	55.8	54.6	49.6	48.2	46.8	44.1
	W850	64.8	63.7	58.9	57.6	56.2	53.5
	W860	63.6	62.5	57.6	56.3	55.0	52.2
	W870	69.5	65.9	60.6	58.0	55.0	51.8
	W880	68.0	66.9	62.3	61.1	59.7	57.0
	W890	69.9	67.0	61.9	59.4	56.5	54.1
	W900	66.4	66.0	61.5	59.4	57.0	52.8
	W910	65.6	68.7	65.2	61.5	57.1	54.3
	W920	66.7	69.8	66.3	62.8	58.3	55.6
	W930	59.9	63.3	59.5	55.7	51.1	48.3
	W940	67.1	70.2	66.7	63.2	58.8	56.1
	W950	65.4	68.5	64.9	61.3	56.8	54.1
	W960	63.4	68.6	66.2	62.8	60.6	54.9
Plank Creek	W280	70.1	68.4	61.9	58.8	55.1	51.3
Shaytown Branch	W290	60.6	66.0	60.6	56.5	51.4	47.0
	W300	60.2	62.3	55.2	49.7	46.9	43.4
	W690	58.9	67.0	62.4	57.5	53.9	49.6
	W700	62.6	64.9	60.9	58.2	55.1	49.1
South Branch Oswayo Creek	W030	66.0	64.7	60.0	56.7	52.8	48.1

Hydrologic Parameters for Oswayo Creek HEC-HMS Model

Subwatershed Name	Subbasin	Existing CN (ARC=2)	Calibrated Existing Conditions (Year 2010) Curve Numbers				
			2-Year	10-Year	25-Year	50-Year	100-Year
South Branch Oswayo Creek	W380	65.4	64.9	60.3	56.8	52.7	49.0
	W390	62.8	64.6	59.3	55.8	51.7	47.3
	W400	61.2	65.3	59.7	56.2	52.0	48.3
	W410	61.3	63.4	57.9	54.5	50.4	47.3
Whitney Creek	W080	59.2	64.4	58.3	55.3	51.7	47.5
	W090	58.4	65.9	61.0	57.2	54.5	49.3
	W100	59.8	58.6	51.8	50.3	48.8	43.0
Wildcat Creek	W560	70.3	65.5	60.7	58.0	54.8	51.5

Hydrologic Parameters for Oswayo Creek HEC-HMS Model

Subwatershed Name	Subbasin	Future CN (ARC=2)	Calibrated Future Conditions (Year 2020) Curve Numbers				
			2-Year	10-Year	25-Year	50-Year	100-Year
Bell Run	W310	60.4	67.1	62.2	58.3	54.8	49.2
	W710	61.8	67.1	61.8	58.5	54.6	48.5
	W720	59.7	66.7	61.8	57.5	53.8	47.5
	W730	60.8	68.5	64.0	59.2	55.7	52.0
Bradley Run	W110	58.0	63.5	61.0	56.6	53.3	48.1
	W120	63.5	62.4	57.6	55.3	52.7	46.9
	W130	63.3	63.9	59.1	56.1	52.5	48.1
	W140	63.5	64.1	59.3	56.3	52.7	48.3
	W150	61.6	60.5	55.6	53.3	50.7	48.1
	W160	62.2	61.1	56.2	53.9	51.3	48.2
Butter Creek	W250	70.2	67.7	59.7	56.2	52.1	49.8
	W260	65.9	66.2	60.8	57.7	53.9	50.1
Canada Run	W530	62.9	64.9	59.5	56.1	52.0	48.1
	W540	64.7	64.1	60.2	58.1	55.7	50.6
Clara Creek	W010	63.5	62.8	58.1	54.6	50.4	47.2
	W020	61.9	60.7	55.9	53.5	51.0	45.2
	W420	63.6	62.5	57.7	55.4	52.8	47.0
	W430	67.7	66.7	60.2	56.7	52.4	51.5
	W440	65.8	64.7	58.1	54.5	50.2	44.7
	W450	64.3	64.9	60.1	57.1	53.5	47.7
Cow Run	W570	65.0	64.5	60.6	58.0	55.0	50.7
Elevenmile Creek	W170	71.0	65.1	61.2	57.9	53.9	51.3
	W180	70.7	64.3	60.3	57.2	53.6	51.1
	W470	72.4	64.3	60.0	56.8	53.1	52.8
	W480	69.3	64.1	58.2	55.3	51.9	51.6
	W490	68.4	66.0	61.0	57.5	53.2	48.0
	W500	67.0	63.4	58.0	55.4	52.4	47.7
	W510	65.8	64.7	60.0	57.8	55.2	49.4
	W520	68.9	64.4	58.7	55.9	52.6	49.9
Hemlock Hollow Run	W040	66.1	65.0	60.3	57.3	53.7	49.7
	W050	69.9	66.1	58.5	56.8	54.9	54.1
	W060	67.3	63.3	59.8	56.2	51.9	51.1
	W070	67.8	65.8	61.8	58.0	53.3	48.9
	W330	69.5	64.8	58.9	55.1	50.6	48.9
	W340	66.8	63.9	57.9	54.5	50.5	47.5
	W350	64.3	63.2	58.4	56.1	53.6	47.7
Honeoye Creek	W190	69.8	67.0	58.8	55.9	52.5	49.3
	W200	70.3	67.9	60.0	57.2	54.0	49.9
	W210	67.4	70.5	65.6	58.9	54.6	46.5
	W220	69.4	67.9	61.3	57.9	53.8	48.8
	W230	66.7	69.3	63.1	59.2	54.3	49.5
	W240	66.6	69.5	63.2	59.3	54.5	49.7
	W270	67.4	68.0	60.8	57.4	53.4	50.0
	W580	68.5	67.1	61.6	57.5	52.4	49.4

Hydrologic Parameters for Oswayo Creek HEC-HMS Model

Subwatershed Name	Subbasin	Future CN (ARC=2)	Calibrated Future Conditions (Year 2020) Curve Numbers				
			2-Year	10-Year	25-Year	50-Year	100-Year
Honeoye Creek	W590	66.7	64.4	58.3	52.9	48.6	45.7
	W600	68.5	69.1	66.8	63.9	60.4	52.5
	W610	67.7	66.5	62.7	57.1	52.4	48.5
	W620	67.3	68.9	64.7	61.1	56.6	54.3
	W630	69.9	68.9	65.3	63.7	61.9	54.1
	W640	64.9	64.8	61.1	58.1	54.6	50.0
	W650	68.8	66.0	61.9	58.2	53.6	49.7
	W660	71.6	69.6	65.8	63.6	61.1	58.5
Horse Run	W680	67.7	70.8	67.3	60.4	56.2	52.4
Janders Run	W670	59.8	66.5	62.9	58.9	56.3	49.6
Oswayo Creek	W320	68.7	65.2	59.9	56.1	51.5	49.0
	W360	62.5	66.3	60.2	56.7	52.5	49.3
	W370	60.5	64.7	59.1	55.4	50.9	48.4
	W460	61.2	65.1	58.8	56.0	52.8	47.9
	W550	61.9	64.2	60.2	58.3	56.2	50.1
	W740	62.2	69.2	63.1	60.4	57.9	52.8
	W750	70.9	69.9	61.8	59.0	55.6	55.3
	W760	68.2	64.2	58.7	55.4	51.5	49.0
	W770	65.1	63.8	59.0	55.2	50.5	45.9
	W780	65.3	64.1	59.3	55.5	50.8	47.7
	W790	76.0	75.1	67.8	65.3	62.4	61.6
	W800	90.9	90.5	87.9	86.7	85.4	85.0
	W810	61.9	65.0	59.1	55.8	51.9	48.4
	W820	61.9	64.7	58.9	55.6	51.6	48.3
	W830	60.7	64.3	58.2	55.4	52.3	46.8
	W840	55.8	54.6	49.6	48.2	46.8	44.1
	W850	64.8	63.7	58.9	57.6	56.2	53.5
	W860	64.9	63.8	59.1	57.8	56.4	53.6
	W870	69.6	66.0	60.7	58.1	55.1	51.9
	W880	68.0	66.9	62.3	61.1	59.7	57.0
	W890	69.9	67.0	61.9	59.4	56.5	54.1
	W900	67.2	66.8	62.4	60.2	57.8	53.7
	W910	70.6	73.5	70.2	66.8	62.6	60.0
	W920	72.7	75.4	72.3	69.1	65.0	62.4
	W930	60.1	63.5	59.6	55.8	51.2	48.5
	W940	67.1	70.2	66.7	63.2	58.8	56.1
	W950	65.4	68.5	64.9	61.3	56.8	54.1
	W960	63.4	68.6	66.2	62.8	60.6	54.9
Plank Creek	W280	70.1	68.4	61.9	58.8	55.1	51.3
Shaytown Branch	W290	60.6	66.0	60.6	56.5	51.4	47.0
	W300	60.2	62.3	55.2	49.7	46.9	43.4
	W690	58.9	67.0	62.4	57.5	53.9	49.6
	W700	62.6	64.9	60.9	58.2	55.1	49.1
South Branch Oswayo Creek	W030	66.0	64.7	60.0	56.7	52.8	48.1

Hydrologic Parameters for Oswayo Creek HEC-HMS Model

Subwatershed Name	Subbasin	Future CN (ARC=2)	Calibrated Future Conditions (Year 2020) Curve Numbers				
			2-Year	10-Year	25-Year	50-Year	100-Year
South Branch Oswayo Creek	W380	65.4	64.9	60.3	56.8	52.7	49.0
	W390	63.0	64.7	59.4	56.0	51.9	47.4
	W400	61.4	65.5	59.9	56.4	52.2	48.5
	W410	61.8	63.8	58.4	54.9	50.9	47.8
Whitney Creek	W080	59.2	64.4	58.3	55.3	51.7	47.5
	W090	58.4	65.9	61.0	57.2	54.5	49.3
	W100	59.8	58.6	51.8	50.3	48.8	43.0
Wildcat Creek	W560	70.3	65.5	60.7	58.0	54.8	51.5

Hydrologic Results for Oswayo Creek HEC-HMS Model

Discharge Point	HEC-HMS Node	Coordinates		Cumulative Area (mi^2)	2010 Discharges with Existing SWM					2020 Discharges with No Future SWM				
		x	y		2-Year	10-Year	25-Year	50-Year	100-Year	2-Year	10-Year	25-Year	50-Year	100-Year
1	J713	1903205.5	649030.4	2.37	106	211	291	311	454	105	211	291	310	453
2	J706	1915162.6	650545.5	1.81	99	189	300	386	514	99	189	300	386	327
3	O10	1913195.3	649561.8	2.25	115	233	362	456	624	115	233	362	456	395
4	J553	1898894.0	641314.0	10.52	427	893	1,251	1,417	1,825	426	893	1,251	1,417	1,593
5	J583	1899802.1	620505.3	6.32	217	492	689	778	880	217	492	689	778	880
6	J589	1895035.0	623456.4	9.60	330	715	1,018	1,166	1,298	332	741	1,039	1,172	1,305
7	J604	1884365.8	627315.4	6.68	231	477	669	827	878	231	477	669	827	878
8	J599	1884668.5	629055.8	19.57	654	1,378	1,928	2,259	2,487	659	1,410	1,957	2,271	2,501
9	O17	1866659.5	628828.8	1.02	42	133	169	206	207	42	133	169	206	207
10	P71	1867113.5	621867.3	1.05	46	112	166	198	215	46	112	166	198	215
11	P70	1869459.2	625423.7	2.32	102	243	361	428	466	102	243	361	428	466
12	J609	1871275.3	627920.8	4.37	168	440	633	768	826	168	440	633	768	826
13	J586	1873621.0	623305.0	1.99	77	202	312	378	408	77	202	312	378	408
14	J594	1872410.3	624061.7	3.01	105	281	448	559	576	105	281	448	559	576
15	J612	1872713.0	628753.1	7.89	287	744	1,111	1,361	1,437	287	744	1,111	1,361	1,437
16	J_Clara Creek	1873993.4	636146.4	9.64	353	896	1,328	1,616	1,682	353	896	1,328	1,616	1,682
17	J556	1900357.0	656775.5	1.63	70	172	250	292	463	70	172	250	292	463
18	J737	1890646.2	653042.5	4.75	181	396	582	693	1,104	181	396	582	693	1,104
19	J716	1882625.4	649561.8	9.50	364	809	1,143	1,320	1,826	364	809	1,143	1,320	1,826
20	J696	1880355.4	648048.4	11.41	409	908	1,285	1,492	2,007	409	908	1,285	1,492	2,007
21	J562	1876799.0	645702.7	13.97	473	1,064	1,504	1,750	2,299	473	1,064	1,504	1,750	2,299
22	O18	1864843.5	636017.2	1.36	76	164	235	265	305	76	164	235	265	305
23	J751	1874924.8	662501.0	2.80	174	257	360	402	542	174	257	360	402	542
24	J763	1892083.9	667873.4	3.64	221	326	489	594	688	221	326	489	594	688
25	J775	1890494.9	671581.1	11.15	498	875	1,178	1,287	1,545	498	875	1,178	1,287	1,545
26	J783	1884668.5	671732.5	19.18	795	1,352	1,764	1,904	2,166	795	1,352	1,764	1,904	2,166
27	J778	1884592.8	671656.8	20.48	832	1,414	1,848	2,000	2,267	832	1,414	1,848	2,000	2,267
28	J772	1869534.9	667949.1	34.97	1,273	2,196	2,795	3,022	3,380	1,273	2,196	2,795	3,022	3,380
29	J559	1864994.8	664695.4	41.07	1,376	2,368	3,016	3,272	3,693	1,376	2,368	3,016	3,273	3,693
30	J756	1864843.5	664392.7	42.71	1,406	2,416	3,079	3,345	3,778	1,407	2,417	3,080	3,346	3,780
31	J734	1860000.8	659474.3	46.49	1,470	2,530	3,230	3,521	3,980	1,471	2,531	3,232	3,523	3,982
32	J740	1855839.0	658339.2	48.43	1,493	2,573	3,285	3,583	4,053	1,494	2,574	3,287	3,585	4,055
33	J_Honeoye Creek	1847386.8	656803.9	50.89	1,430	2,479	3,169	3,470	3,949	1,437	2,488	3,180	3,482	3,962
34	P101	1852812.3	631628.5	1.23	78	163	214	215	235	78	163	214	215	235

Hydrologic Results for Oswayo Creek HEC-HMS Model

Discharge Point	HEC-HMS Node	Coordinates		Cumulative Area (mi^2)	2010 Discharges with Existing SWM					2020 Discharges with No Future SWM				
		x	y		2-Year	10-Year	25-Year	50-Year	100-Year	2-Year	10-Year	25-Year	50-Year	100-Year
35	J624	1852812.3	633898.5	3.27	177	372	461	513	571	177	372	461	513	571
36	J645	1845396.9	637908.9	8.18	414	884	1,166	1,361	1,395	414	884	1,166	1,361	1,395
37	J671	1838813.7	643130.0	11.39	565	1,162	1,568	1,832	1,839	565	1,162	1,568	1,832	1,839
38	J693	1837527.4	646535.1	16.11	765	1,555	2,077	2,433	2,395	765	1,555	2,077	2,433	2,395
39	P35	1919097.4	632233.8	0.81	96	136	198	238	366	96	136	198	238	366
40	J627	1913497.9	634503.9	2.79	182	320	466	545	778	182	320	466	545	778
41	J653	1908352.5	638892.6	7.11	358	705	984	1,095	1,436	358	705	984	1,095	1,436
42	J661	1898667.0	640179.0	19.03	824	1,680	2,340	2,624	3,401	824	1,680	2,340	2,624	3,172
43	J674	1895943.0	639270.9	20.96	887	1,791	2,487	2,788	3,603	895	1,804	2,502	2,806	3,400
44	J658	1895716.0	639119.6	23.95	979	1,980	2,733	3,058	3,961	992	2,001	2,763	3,087	3,778
45	J637	1886484.5	634503.9	27.90	1,073	2,161	2,976	3,327	4,270	1,093	2,192	3,017	3,369	4,116
46	J_S.Branch Oswayo Creek	1881793.1	633595.9	50.61	1,772	3,599	4,969	5,639	6,843	1,796	3,662	5,042	5,698	6,732
47	J634	1874075.0	636395.6	62.83	1,989	4,013	5,532	6,308	7,523	2,014	4,080	5,612	6,375	7,442
48	J640	1873848.0	637227.9	63.94	2,007	4,046	5,576	6,361	7,580	2,033	4,113	5,656	6,428	7,500
49	J_Elevenmile Creek and Osway	1869534.9	639573.6	79.92	2,379	4,780	6,644	7,625	9,188	2,407	4,858	6,742	7,711	9,184
50	J648	1866810.9	641011.3	83.91	2,420	4,870	6,771	7,787	9,356	2,452	4,951	6,873	7,880	9,366
51	J683	1860303.4	644794.7	86.83	2,445	4,921	6,841	7,880	9,446	2,479	5,004	6,945	7,977	9,465
52	J686	1859395.4	645248.7	90.47	2,513	5,052	7,024	8,102	9,709	2,547	5,136	7,130	8,201	9,735
53	J699	1853190.6	649637.4	95.50	2,577	5,174	7,192	8,314	9,948	2,613	5,260	7,301	8,417	9,984
54	USGS 03010655	1845850.9	654404.5	97.67	2,579	5,182	7,201	8,336	9,954	2,620	5,271	7,314	8,444	9,996
55	J721	1846910.2	656901.6	149.35	3,894	7,518	10,182	11,599	13,647	3,944	7,615	10,301	11,715	13,690
56	J724	1844715.8	657052.9	150.90	3,824	7,408	10,034	11,442	13,438	3,874	7,506	10,153	11,560	13,483
57	J729	1839721.8	660079.6	159.06	4,004	7,755	10,415	11,876	13,943	4,056	7,855	10,536	11,998	13,996
58	J_Bell Run	1837019.2	662099.8	20.51	934	1,870	2,448	2,869	2,906	934	1,870	2,448	2,869	2,906
59	J743	1836997.7	662274.0	179.85	4,385	8,429	11,292	12,925	15,076	4,440	8,533	11,420	13,055	15,149
60	J748	1836922.0	662349.6	180.88	4,392	8,445	11,313	12,953	15,107	4,447	8,548	11,441	13,083	15,181
61	Outlet-Oswayo	1832117.1	665943.9	182.07	4,256	8,249	11,050	12,676	14,736	4,310	8,350	11,174	12,803	14,808

Hydrologic Parameters for Genesee River HEC-HMS Model

Subwatershed Name	Subbasin	Drainage Area (mi^2)	Existing Conditions (2010)		Future Conditions (2020)	
			CN	Lag (min)	CN	Lag (min)
Ainsworth Brook	W230	1.09	56.4	46.1	56.4	57.7
Cotton Brook	W430	2.57	56.0	93.6	56.3	92.8
Cryder Creek	W590	16.04	59.0	237.9	59.1	215.5
Ellisburg Creek	W050	2.95	56.4	65.3	56.4	81.5
Genesee River	W730	2.06	56.5	63.6	56.5	79.5
	W740	0.04	55.1	19.5	60.1	19.1
	W750	0.41	53.5	31.4	61.8	31.8
	W760	0.33	61.6	37.7	70.8	37.1
	W770	0.07	71.6	27.4	82.6	24.6
	W780	1.34	55.6	61.7	56.2	75.9
	W790	1.92	56.6	78.3	56.6	97.8
	W800	5.24	57.1	98.7	57.1	123.4
	W810	0.26	58.1	38.7	58.1	48.4
	W820	0.25	57.3	26.8	57.3	26.8
Gennessee River	W670	3.85	61.7	98.8	63.3	118.6
	W680	2.50	59.1	71.8	59.1	89.7
	W690	0.27	59.0	31.1	59.0	38.8
	W700	0.17	60.8	18.2	60.8	22.7
	W710	0.20	60.2	22.2	60.2	27.8
	W720	1.02	56.5	59.1	56.5	73.9
Irish Settlement Brook	W240	2.26	58.3	60.1	58.3	75.2
	W250	1.35	56.0	58.2	56.0	72.8
Leadville Hollow	W220	2.74	55.5	72.7	55.5	80.8
Ludington Run	W350	1.19	58.6	55.4	58.6	69.2
	W360	0.30	62.8	36.6	62.8	45.7
	W370	3.95	57.1	74.1	57.1	92.6
	W380	1.09	56.9	48.4	56.9	60.4
Marsh Creek	W580	15.26	57.6	162.4	57.6	203.0
	W650	2.02	56.9	69.8	56.9	87.3
Middle Branch	W440	1.69	59.5	70.6	59.7	87.9
	W450	3.63	56.5	89.1	56.8	110.4
	W460	1.22	55.8	43.1	55.8	53.8
	W470	2.42	56.5	61.0	56.5	76.3
	W480	3.39	56.3	78.3	56.3	97.9
Mundy Brook	W490	2.13	60.3	64.6	60.3	80.7
	W500	0.93	56.6	37.1	56.6	46.4
	W510	1.70	57.1	54.3	57.5	67.3
Orebed Creek	W300	2.76	59.0	76.2	59.0	95.2
	W310	1.55	55.7	64.4	55.7	80.5
Redwater Creek	W630	1.49	57.6	62.5	57.6	78.2
	W640	2.03	55.9	62.9	55.9	78.6
Rose Brook	W020	1.20	61.4	65.7	61.4	82.2
	W030	2.09	59.8	73.3	59.8	91.6
Rose Lake Run	W180	1.81	57.4	71.9	58.0	88.6

Hydrologic Parameters for Genesee River HEC-HMS Model

Subwatershed Name	Subbasin	Drainage Area (mi^2)	Existing Conditions (2010) CN	Existing Conditions (2010) Lag (min)	Future Conditions (2020) CN	Future Conditions (2020) Lag (min)
Rose Lake Run	W190	0.15	61.3	26.6	61.3	33.2
	W200	0.88	62.6	51.4	62.4	64.5
	W210	0.00	85.0	2.4	85.0	2.4
Spring Mill Creek	W260	2.07	61.7	69.7	61.7	87.1
	W270	2.95	59.4	87.5	59.4	109.4
	W280	0.95	55.3	52.9	55.3	66.1
Trib to Genesee River	W290	1.03	57.0	58.0	57.0	72.4
	W330	1.68	61.2	67.4	62.0	82.5
	W340	3.14	61.8	80.3	62.5	98.8
	W420	1.09	55.8	53.5	55.8	66.8
	W600	1.15	59.2	45.4	59.2	56.7
	W610	1.04	55.4	56.4	55.4	70.5
	W620	2.76	58.6	89.9	58.6	112.4
	W660	3.14	60.2	88.2	60.2	110.2
Trib to Ludington Run	W090	2.66	61.0	81.7	61.0	102.1
	W100	2.04	61.2	68.8	61.2	86.0
	W110	0.16	65.4	20.9	65.4	26.1
Trib to Middle Branch	W120	1.42	60.5	63.3	60.6	78.8
	W160	0.95	57.1	40.3	57.1	50.4
	W170	1.98	54.8	65.7	54.8	82.1
Trib to Redwater Creek	W320	2.74	60.3	90.9	60.3	113.6
Trib to Rose Brook	W010	0.88	61.6	71.4	61.6	89.2
Trib to Rose Lake Run	W040	0.97	57.8	68.2	57.8	85.2
Trib to Spring Mill Creek	W060	2.77	62.1	84.3	62.4	104.7
Trib to Turner Creek	W140	2.71	58.9	99.6	58.9	124.5
	W150	1.04	59.6	54.3	59.8	67.6
Trib to West Branch	W130	1.44	57.8	57.5	57.8	71.9
Turner Creek	W390	2.46	60.1	72.9	60.2	90.9
	W400	0.09	61.7	19.1	61.7	23.9
	W410	1.83	60.8	60.1	60.8	75.1
West Branch Genesse River	W520	1.62	53.4	64.5	53.4	80.7
	W530	2.37	57.4	82.8	57.3	103.6
	W540	2.68	56.1	49.9	56.1	62.4
	W550	1.73	52.6	51.1	52.6	63.9
	W560	0.49	43.4	38.0	43.4	47.5
	W570	0.36	46.1	48.3	49.5	55.3
Wileyville Creek	W070	4.44	59.8	108.7	59.8	135.8
	W080	1.90	59.3	74.6	59.3	93.2

Hydrologic Parameters for Genesee River HEC-HMS Model

Subwatershed Name	Subbasin	Existing CN (ARC=2)	Calibrated Existing Conditions (Year 2010) Curve Numbers				
			2-Year	10-Year	25-Year	50-Year	100-Year
Ainsworth Brook	W230	56.4	68.9	62.5	58.0	54.6	50.2
Cotton Brook	W430	56.0	70.6	64.3	60.2	55.5	52.3
Cryder Creek	W590	59.0	72.0	67.4	63.6	58.8	55.9
Ellisburg Creek	W050	56.4	65.9	61.2	55.8	52.5	49.3
Genesee River	W730	56.5	69.8	64.0	59.9	55.8	52.1
	W740	55.1	66.5	61.3	56.0	52.6	49.1
	W750	53.5	65.0	59.7	54.4	51.0	47.5
	W760	61.6	72.2	67.4	62.5	59.2	55.8
	W770	71.6	80.2	76.4	72.3	69.5	66.4
	W780	55.6	69.8	64.8	61.0	56.3	53.5
	W790	56.6	71.3	65.3	61.3	56.4	53.5
	W800	57.1	71.8	65.5	61.5	56.8	53.7
	W810	58.1	69.1	64.1	59.0	55.7	52.2
	W820	57.3	68.4	63.3	58.2	54.8	51.3
Gennessee River	W670	61.7	71.9	65.9	61.1	58.0	53.2
	W680	59.1	70.5	64.3	59.4	56.6	52.0
	W690	59.0	69.9	64.9	59.9	56.6	53.1
	W700	60.8	71.5	66.7	61.7	58.4	55.0
	W710	60.2	70.9	66.0	61.0	57.8	54.3
	W720	56.5	70.5	64.7	60.8	56.2	53.3
Irish Settlement Brook	W240	58.3	66.6	61.5	56.9	53.6	49.6
	W250	56.0	68.3	62.8	58.0	54.3	51.1
Leadville Hollow	W220	55.5	67.8	62.3	56.0	52.1	48.5
Ludington Run	W350	58.6	67.1	63.6	58.1	54.9	51.3
	W360	62.8	73.2	68.5	63.7	60.5	57.0
	W370	57.1	67.7	63.1	57.6	54.2	50.4
	W380	56.9	67.5	63.2	58.3	54.8	51.3
Marsh Creek	W580	57.6	70.3	65.2	60.3	56.0	52.9
	W650	56.9	68.5	64.5	60.4	56.2	53.3
Middle Branch	W440	59.5	68.7	64.5	59.1	55.9	53.3
	W450	56.5	69.5	64.5	59.7	55.1	53.1
	W460	55.8	67.2	61.7	56.1	52.9	50.2
	W470	56.5	68.7	63.0	58.1	54.4	51.6
	W480	56.3	69.3	64.4	59.6	55.0	53.3
Mundy Brook	W490	60.3	69.8	63.7	59.0	55.6	50.8
	W500	56.6	67.8	62.7	57.5	54.2	50.6
	W510	57.1	69.5	63.3	59.0	55.6	51.1
Orebed Creek	W300	59.0	69.1	64.2	59.1	56.3	51.7
	W310	55.7	67.5	64.9	61.0	56.0	53.5
Redwater Creek	W630	57.6	68.3	64.3	60.2	56.5	53.3
	W640	55.9	67.6	63.8	59.6	55.2	52.5
Rose Brook	W020	61.4	67.4	64.0	59.6	55.9	52.0
	W030	59.8	67.2	63.7	58.8	55.5	51.6
Rose Lake Run	W180	57.4	68.6	62.7	58.2	55.3	50.5

Hydrologic Parameters for Genesee River HEC-HMS Model

Subwatershed Name	Subbasin	Existing CN (ARC=2)	Calibrated Existing Conditions (Year 2010) Curve Numbers				
			2-Year	10-Year	25-Year	50-Year	100-Year
Rose Lake Run	W190	61.3	71.9	67.1	62.2	59.0	55.5
	W200	62.6	73.0	68.3	63.4	60.2	56.8
	W210	85.0	90.1	87.9	85.5	83.7	81.7
Spring Mill Creek	W260	61.7	67.0	62.9	59.2	55.5	51.3
	W270	59.4	67.7	64.4	59.4	56.7	52.6
	W280	55.3	66.6	61.4	56.2	52.8	49.3
Trib to Genesee River	W290	57.0	70.2	64.6	60.6	56.4	53.1
	W330	61.2	70.5	64.6	59.9	56.5	52.0
	W340	61.8	70.7	64.6	60.1	56.5	51.8
	W420	55.8	70.0	63.9	59.9	55.4	52.3
	W600	59.2	68.8	62.7	58.0	54.7	50.4
	W610	55.4	70.3	64.9	61.1	56.3	53.4
	W620	58.6	71.6	66.1	62.1	57.9	54.3
	W660	60.2	72.0	66.0	61.8	58.6	54.1
Trib to Ludington Run	W090	61.0	68.7	64.9	59.7	56.0	52.2
	W100	61.2	68.3	64.1	59.2	55.2	51.4
	W110	65.4	75.3	70.9	66.3	63.2	59.8
Trib to Middle Branch	W120	60.5	68.4	64.1	59.0	55.3	52.5
	W160	57.1	68.3	63.2	58.0	54.7	51.1
	W170	54.8	66.7	63.4	58.7	53.8	52.3
Trib to Redwater Creek	W320	60.3	70.3	66.1	61.8	58.5	54.0
Trib to Rose Brook	W010	61.6	72.1	67.4	62.4	59.2	55.7
Trib to Rose Lake Run	W040	57.8	68.8	63.8	58.7	55.3	51.8
Trib to Spring Mill Creek	W060	62.1	67.4	63.9	59.7	55.8	51.7
Trib to Turner Creek	W140	58.9	72.5	66.8	62.2	57.7	55.2
	W150	59.6	70.1	63.8	58.7	55.3	51.6
Trib to West Branch	W130	57.8	67.2	61.8	56.8	53.4	50.1
Turner Creek	W390	60.1	70.5	64.3	59.5	56.5	50.9
	W400	61.7	72.2	67.4	62.5	59.3	55.8
	W410	60.8	69.3	63.4	58.9	55.3	50.5
West Branch Genesse River	W520	53.4	66.3	62.1	58.4	54.0	50.3
	W530	57.4	69.8	64.5	59.9	55.9	52.6
	W540	56.1	66.0	60.4	55.1	51.5	48.2
	W550	52.6	65.5	60.9	57.1	52.6	49.2
	W560	43.4	55.3	49.7	44.3	41.0	37.6
	W570	46.1	58.0	52.4	47.0	43.7	40.2
Wileyville Creek	W070	59.8	67.4	65.0	59.7	56.8	52.8
	W080	59.3	67.4	64.3	59.2	56.3	52.5

Hydrologic Parameters for Genesee River HEC-HMS Model

Subwatershed Name	Subbasin	Future CN (ARC=2)	Calibrated Future Conditions (Year 2020) Curve Numbers				
			2-Year	10-Year	25-Year	50-Year	100-Year
Ainsworth Brook	W230	56.4	68.9	62.5	58.0	54.6	50.2
Cotton Brook	W430	56.3	70.9	64.6	60.5	55.9	52.6
Cryder Creek	W590	59.1	72.2	67.6	63.7	59.0	56.1
Ellisburg Creek	W050	56.4	65.9	61.3	55.8	52.5	49.3
Genesee River	W730	56.5	69.8	64.0	59.9	55.8	52.1
	W740	60.1	70.8	65.9	60.9	57.7	54.2
	W750	61.8	72.3	67.5	62.6	59.4	55.9
	W760	70.8	79.6	75.7	71.5	68.6	65.5
	W770	82.6	88.4	85.9	83.1	81.1	78.8
	W780	56.2	70.3	65.4	61.6	56.9	54.2
	W790	56.6	71.3	65.3	61.3	56.4	53.5
	W800	57.1	71.8	65.5	61.5	56.8	53.7
	W810	58.1	69.1	64.1	59.0	55.7	52.2
	W820	57.3	68.4	63.3	58.2	54.8	51.3
Gennessee River	W670	63.3	73.3	67.4	62.7	59.6	54.9
	W680	59.1	70.5	64.3	59.4	56.6	52.0
	W690	59.0	69.9	64.9	59.9	56.6	53.1
	W700	60.8	71.5	66.7	61.7	58.4	55.0
	W710	60.2	70.9	66.0	61.0	57.8	54.3
	W720	56.5	70.5	64.7	60.8	56.2	53.3
Irish Settlement Brook	W240	58.3	66.6	61.5	56.9	53.6	49.6
	W250	56.0	68.3	62.8	58.0	54.3	51.1
Leadville Hollow	W220	55.5	67.8	62.3	56.0	52.1	48.5
Ludington Run	W350	58.6	67.1	63.6	58.1	54.9	51.3
	W360	62.8	73.2	68.5	63.7	60.5	57.0
	W370	57.1	67.7	63.1	57.6	54.2	50.4
	W380	56.9	67.5	63.2	58.3	54.8	51.3
Marsh Creek	W580	57.6	70.3	65.2	60.3	56.0	52.9
	W650	56.9	68.5	64.5	60.4	56.2	53.3
Middle Branch	W440	59.7	68.9	64.6	59.3	56.0	53.4
	W450	56.8	69.8	64.9	60.1	55.5	53.4
	W460	55.8	67.2	61.7	56.1	52.9	50.2
	W470	56.5	68.7	63.0	58.1	54.4	51.6
	W480	56.3	69.3	64.4	59.6	55.0	53.3
Mundy Brook	W490	60.3	69.8	63.7	59.0	55.6	50.8
	W500	56.6	67.8	62.7	57.5	54.2	50.6
	W510	57.5	69.9	63.7	59.4	56.0	51.5
Orebed Creek	W300	59.0	69.1	64.2	59.1	56.3	51.7
	W310	55.7	67.5	64.9	61.0	56.0	53.5
Redwater Creek	W630	57.6	68.3	64.3	60.2	56.5	53.3
	W640	55.9	67.6	63.8	59.6	55.2	52.5
Rose Brook	W020	61.4	67.4	64.0	59.6	55.9	52.0
	W030	59.8	67.2	63.7	58.8	55.5	51.6
Rose Lake Run	W180	58.0	69.1	63.2	58.8	55.9	51.1

Hydrologic Parameters for Genesee River HEC-HMS Model

Subwatershed Name	Subbasin	Future CN (ARC=2)	Calibrated Future Conditions (Year 2020) Curve Numbers				
			2-Year	10-Year	25-Year	50-Year	100-Year
Rose Lake Run	W190	61.3	71.9	67.1	62.2	59.0	55.5
	W200	62.4	72.9	68.2	63.3	60.1	56.7
	W210	85.0	90.1	87.9	85.5	83.7	81.7
Spring Mill Creek	W260	61.7	67.0	62.9	59.2	55.5	51.3
	W270	59.4	67.7	64.4	59.4	56.7	52.6
	W280	55.3	66.6	61.4	56.2	52.8	49.3
Trib to Genesee River	W290	57.0	70.2	64.6	60.6	56.4	53.1
	W330	62.0	71.3	65.3	60.7	57.4	52.8
	W340	62.5	71.3	65.3	60.7	57.2	52.5
	W420	55.8	70.0	63.9	59.9	55.4	52.3
	W600	59.2	68.8	62.7	58.0	54.7	50.4
	W610	55.4	70.3	64.9	61.1	56.3	53.4
	W620	58.6	71.6	66.1	62.1	57.9	54.3
	W660	60.2	72.0	66.0	61.8	58.6	54.1
Trib to Ludington Run	W090	61.0	68.7	64.9	59.7	56.0	52.2
	W100	61.2	68.3	64.1	59.2	55.2	51.4
	W110	65.4	75.3	70.9	66.3	63.2	59.8
Trib to Middle Branch	W120	60.6	68.5	64.3	59.2	55.4	52.6
	W160	57.1	68.3	63.2	58.0	54.7	51.1
	W170	54.8	66.7	63.4	58.7	53.8	52.3
Trib to Redwater Creek	W320	60.3	70.3	66.1	61.8	58.5	54.0
Trib to Rose Brook	W010	61.6	72.1	67.4	62.4	59.2	55.7
Trib to Rose Lake Run	W040	57.8	68.8	63.8	58.7	55.3	51.8
Trib to Spring Mill Creek	W060	62.4	67.7	64.2	59.9	56.0	52.0
Trib to Turner Creek	W140	58.9	72.5	66.8	62.2	57.7	55.2
	W150	59.8	70.2	64.0	58.9	55.5	51.8
Trib to West Branch	W130	57.8	67.2	61.8	56.8	53.4	50.1
Turner Creek	W390	60.2	70.6	64.4	59.6	56.6	51.0
	W400	61.7	72.2	67.4	62.5	59.3	55.8
	W410	60.8	69.3	63.4	58.9	55.3	50.5
West Branch Genesse River	W520	53.4	66.3	62.1	58.4	54.0	50.3
	W530	57.3	69.7	64.5	59.9	55.9	52.6
	W540	56.1	66.0	60.5	55.1	51.6	48.2
	W550	52.6	65.5	60.9	57.1	52.6	49.2
	W560	43.4	55.3	49.7	44.3	41.0	37.6
	W570	49.5	61.3	55.8	50.5	47.1	43.6
Wileyville Creek	W070	59.8	67.4	65.0	59.7	56.8	52.8
	W080	59.3	67.4	64.3	59.2	56.3	52.5

Hydrologic Results for Genesee River HEC-HMS Model

Discharge Point	HEC-HMS Node	Coordinates		Cumulative Area (mi^2)	2010 Discharges with Existing SWM					2020 Discharges with No Future SWM				
		x	y		2-Year	10-Year	25-Year	50-Year	100-Year	2-Year	10-Year	25-Year	50-Year	100-Year
1	P88	1968584.2	637530.6	1.68	134	229	278	333	363	146	248	302	361	395
2	J495	1955342.3	627542.4	3.85	211	466	540	635	742	211	466	540	636	742
3	P76	1952769.6	627693.8	2.04	124	262	314	355	413	124	262	314	355	413
4	J503	1954282.9	630190.8	6.35	363	778	912	1,059	1,236	363	778	912	1,059	1,236
5	P81	1952164.2	641692.3	10.31	554	1,157	1,340	1,574	1,833	554	1,157	1,340	1,574	1,833
6	J_Ludington Run	1951480.6	645592.6	11.39	593	1,232	1,429	1,679	1,957	593	1,232	1,429	1,679	1,957
7	J562	1960714.7	650242.8	3.50	266	449	535	653	685	271	455	543	662	695
8	J552	1958293.3	648956.4	6.30	449	755	908	1,057	1,204	453	760	915	1,065	1,212
9	J490	1944294.8	620732.3	3.11	198	414	480	569	722	201	420	487	578	731
10	J513	1943689.4	633141.8	7.70	439	861	1,016	1,157	1,493	450	880	1,038	1,182	1,524
11	J523	1940284.4	639270.9	10.90	546	1,071	1,266	1,432	1,860	558	1,090	1,289	1,459	1,894
12	O8	1941041.0	645702.7	13.32	641	1,223	1,447	1,641	2,121	653	1,243	1,471	1,668	2,156
13	J_Middle Branch Creek	1939097.7	658537.3	16.71	775	1,444	1,716	1,935	2,521	787	1,464	1,741	1,964	2,557
14	J590	1950348.2	666360.0	3.22	243	418	513	616	664	243	418	513	616	664
15	J603	1948305.2	665073.7	4.15	301	531	645	774	853	301	531	645	774	853
16	J508	1931506.9	631325.8	3.06	164	349	444	508	606	164	349	444	508	606
17	P56	1924242.8	660457.9	2.26	121	250	310	375	429	121	250	310	375	429
18	P36	1926210.1	634806.5	1.81	111	198	247	309	328	119	211	262	327	349
19	J518	1926437.1	636471.2	2.94	181	332	404	497	552	189	345	420	517	573
20	J530	1929161.2	642600.3	12.21	683	1,327	1,610	1,899	2,242	691	1,340	1,626	1,918	2,263
21	O9	1931960.9	651831.8	14.89	771	1,486	1,792	2,113	2,501	779	1,499	1,808	2,133	2,522
22	J572	1932414.9	658944.6	19.35	937	1,786	2,128	2,494	2,956	944	1,799	2,144	2,514	2,977
23	J585	1934306.6	661971.3	23.45	1,087	2,067	2,459	2,885	3,418	1,094	2,078	2,473	2,904	3,438
24	J_W.Branch Genessee	1935945.2	664885.3	23.81	1,090	2,075	2,467	2,896	3,430	1,100	2,089	2,485	2,918	3,455
25	J621	1976605.0	669765.1	2.09	147	303	368	434	507	147	303	368	434	507
26	J639	1974410.6	676499.5	8.62	410	948	1,123	1,357	1,563	410	948	1,123	1,357	1,563
27	J595	1963287.4	669159.8	4.83	235	531	686	791	894	240	541	698	805	909
28	J628	1967524.8	679753.3	18.30	866	1,987	2,409	2,898	3,309	872	1,997	2,422	2,912	3,325
29	J656	1963665.7	682780.0	34.51	1,415	2,971	3,585	4,196	4,907	1,420	2,981	3,596	4,209	4,922
30	J_Cryder Creek	1936487.4	667185.8	50.54	1,968	3,854	4,697	5,357	6,346	1,980	3,873	4,721	5,384	6,378
31	J609	1926664.2	667722.1	2.18	183	325	408	474	547	183	325	408	474	547
32	J100	1916676.0	667343.7	2.76	172	332	392	488	533	172	332	392	488	533

Hydrologic Results for Genesee River HEC-HMS Model

Discharge Point	HEC-HMS Node	Coordinates		Cumulative Area (mi^2)	2010 Discharges with Existing SWM					2020 Discharges with No Future SWM				
		x	y		2-Year	10-Year	25-Year	50-Year	100-Year	2-Year	10-Year	25-Year	50-Year	100-Year
33	J634	1911303.6	672867.5	4.23	255	521	652	772	879	255	521	652	772	879
34	J646	1917130.0	681039.6	10.57	583	1,192	1,472	1,723	2,000	583	1,192	1,472	1,723	2,000
35	J_Marsh Creek	1919733.2	686697.9	12.59	633	1,286	1,591	1,845	2,161	633	1,286	1,591	1,845	2,161
36	J500	1960563.4	639346.6	8.67	618	1,033	1,251	1,495	1,618	693	1,148	1,391	1,660	1,804
37	P83	1953753.3	645173.0	11.17	756	1,258	1,520	1,826	1,982	835	1,379	1,668	2,002	2,180
38	J539	1951527.7	645969.5	22.83	1,352	2,491	2,952	3,511	3,944	1,432	2,617	3,105	3,693	4,149
39	J544	1949969.9	648199.8	31.14	1,886	3,382	4,025	4,760	5,361	1,969	3,512	4,186	4,951	5,577
40	J547	1948229.5	650091.5	31.34	1,890	3,387	4,031	4,767	5,369	1,973	3,517	4,192	4,957	5,584
41	J555	1947245.8	651529.1	33.44	1,982	3,535	4,216	4,973	5,619	2,065	3,666	4,378	5,164	5,835
42	J567	1940511.4	658793.3	38.07	2,190	3,862	4,626	5,424	6,157	2,274	3,995	4,791	5,621	6,379
43	J577	1939300.7	659171.6	54.82	2,962	5,303	6,338	7,349	8,672	3,059	5,455	6,528	7,574	8,931
44	J582	1938165.7	661820.0	61.08	3,184	5,658	6,774	7,866	9,252	3,287	5,816	6,970	8,100	9,522
45	J598	1936047.0	665452.0	85.23	4,147	7,563	9,072	10,565	12,450	4,265	7,732	9,283	10,819	12,746
46	J606	1935895.6	667192.4	135.84	4,559	8,303	10,148	11,790	13,951	4,684	8,489	10,291	11,959	14,154
47	J614	1931204.2	670900.1	140.41	4,642	8,431	10,212	11,884	14,084	4,765	8,614	10,403	12,050	14,284
48	J631	1926966.8	675213.2	145.09	4,793	8,674	10,479	12,111	14,365	4,916	8,854	10,701	12,339	14,592
49	J651	1920005.4	687017.4	162.92	5,237	9,448	11,430	13,169	15,590	5,354	9,614	11,634	13,412	15,871
50	J665	1920081.0	689514.4	166.32	5,330	9,596	11,613	13,383	15,831	5,447	9,761	11,815	13,625	16,111
51	Outlet-Genessee	1920345.9	690989.9	166.57	5,100	9,277	11,253	13,007	15,443	5,220	9,442	11,456	13,227	15,688

Appendix B – Supporting Calculations for the Design Example

The *Model Ordinance* has been developed to implement a variety of control standards in order to achieve a holistic approach to stormwater management. The overall design process has been addressed in *Section VIII* of this Plan. The following example calculations have been provided to further clarify the design method. These calculations parallel the calculations that are made on the worksheets provided in the *Pennsylvania Stormwater Best Management Practices Manual* (PA BMP Manual) a copy of which are provided at the back of this appendix.

SUPPORTING CALCULATIONS - DESIGN EXAMPLE 1

NON-STRUCTURAL BMP CREDITS

<u>*Protect Sensitive Natural Resources*</u>
(Refer to Worksheet 2 & Worksheet 3)

Stormwater Management Area = Total Drainage Area – Protected Area

= 9.78 – 1.31(woods) – 0.37 (minimum disturbance)
= **8.1-Acres**

This is the total area used for pre-development and post-development volume calculations.

<u>*Minimum Soil Compaction*</u>
(Refer to *Worksheet 3*)

Lawn Area (post development) protected from compaction = 16,165-ft^2

16,165-ft^2 x 1/4" x 1/12 = **337-ft^3**

To be eligible for this credit, areas must not be compacted during construction and be guaranteed to remain protected from compaction. Minimum soil compaction credits for lawn area (Open Space) are applicable for this example because specific measures were utilized to protect the back yard lawn areas of Lots 9 & 10 and this area has been placed in a permanent minimum soil compaction easement. Credits for the meadow area can be applied for areas that are not disturbed during construction and will remain in pre-development vegetated cover condition.

<u>*Disconnect Non-Roof Impervious to Vegetated Areas*</u>
(Refer to *Worksheet 3*)

Lot Impervious Area = 10 (Lots) x 1,000 (ft^2/lot) = 10,000-ft^2.
10,000-ft^2 x 1/3" x 1/12 = **278-ft^3**

This credit is applied for the impervious surfaces (driveways and sidewalks) which direct runoff to vegetated surfaces and not directly into a stormwater collection system. The 1/3" credit is used because runoff discharges across the lawn area and is received by rain gardens, which are structures specifically placed to receive and infiltrate runoff. The 1/4" credit would be used for runoff not discharged to a specific infiltration structure or an area that has been protected from soil compaction.

Summation of Non-Structural BMP Credits
= 337-ft^3 + 278-ft^3 = **615-ft^3**

CHANGE IN RUNOFF VOLUME FOR THE 2-YEAR STORM EVENT

(Refer to *Worksheet 4*)

2-year, 24-hour Rainfall Depth = 2.76"

Pre-Development 2-yr Runoff Volume = 5,682 ft^3

Post-Development 2-yr Runoff Volume = 18,281 ft^3

Change in Runoff Volume for the 2-year, 24-hour storm event:

= 18,2813-ft^3 – 5,682-ft^3 = **12,599-ft^3**

This is the volume that must be managed through a combination of non-structural BMP credits and structural BMP credits.

25% LIMIT FOR NON-STRUCTURAL BMP CREDITS

(Refer to *Worksheet 5*)

*Per Chapter 8 of the Pennsylvania Stormwater BMP Manual, Non-Structural Credits may be **no greater than 25%** of the total required control volume.*

Check 25% Non-Structural Credit Limit:

= 615-ft^3 / 12,599-ft^3 = **4.9%**

Calculated credits are under the allowable 25% limit for non-structural credits.

STRUCTURAL CONTROL VOLUME REQUIREMENT

(Refer to *Worksheet 5*)

Required Structural BMP infiltration volume:

= Change in Runoff Volume – Non-Structural BMP Credits
= 12,599-ft^3 – 615-ft^3 = **11,984-ft^3**

STRUCTURAL BMP VOLUME CREDITS

The sizing of structural infiltration BMPs is based on two primary criteria:

1. Maximum loading ratios – There are two different loading ratios that are important when determining the size of a structural BMP. These ratios are derived from guidelines found in the *Pennsylvania Stormwater BMP Manual*.
    a. Maximum loading ratio of Impervious Area to Infiltration Area = 5:1
    b. Maximum loading ratio of Total Drainage Area to Infiltration Area = 8:1

2. Expected runoff volume loading – Structural BMPs must be sized to accommodate the runoff volume they are expected to receive from the contributing drainage area. Some of this volume will be removed and the remainder must be safely conveyed through an overflow device. The removed volume, or infiltration volume, is the important component for sizing the infiltration BMP. A good starting point for infiltration volume is to calculate the contributing area runoff volume for the 2-year, 24-hour design storm. This volume may not be suitable for a particular site design, but starting with this volume will usually result in a design that is close to what is appropriate, and it can be adjusted as necessary. Additional design restrictions may exist for certain BMPs, so these should be considered prior to using this sizing method.

Dry Wells
(Example calculations shown for Lot #1; Refer to *Worksheet 5A* for additional calculations)

Surface Area:
Find the minimum dry well surface area for each lot based on the maximum loading ratios.

Maximum impervious area to infiltration area loading ratio = 5:1 (3:1 for Karst areas)
Tributary impervious area = 2,150-ft^2 (typ.)
= 2,150-ft^2 / 5 = **430-ft^2**
= minimum surface area of dry well per impervious loading ratio

Maximum total drainage area to infiltration area loading ratio = 8:1
Total drainage area = 2,590-ft^2 (typ.)
= 2,590-ft^2 / 8 = **324-ft^2**
= minimum surface area of dry well per pervious loading ratio

The larger of the two calculated areas is the total minimum surface area required for each lot. An individual dry well is placed at each of the four major corners of the house to promote distribution of impervious area runoff. However, the total surface area is used throughout the remaining volume credit calculations for simplicity. The surface area of each dry well is calculated below:

Total Minimum Dry Well Surface Area ÷ Number of Dry Wells
=430 ft^2 / 4 = **107.5-ft^2**

Each dry well will be 10' x 11' to meet the minimum surface area requirements.

Volume:
Find the infiltration volume for each dry well based on the expected runoff volume.

Land Use	Soil Type	Area	Area	CN	S	I_a	Runoff Depth$_{2\text{-yr}}$	Runoff Volume$_{2\text{-yr}}$
	(HSG)	(sf)	(acres)			(0.2*S)	(in)	(ft^3)
Open Space (good)	B	110	0.00	61	6.393	1.279	0.28	3
Impervious	B	540	0.01	98	0.204	0.041	2.53	114
TOTAL:		**650**	**0.01**				**2.81**	**116**

Runoff volume = **116-ft^3**

Depth:
Each dry well will be filled with aggregate. The in-place aggregate will have a 40% voids ratio; therefore the volume is divided by the available void space to get a total volume.

Depth = Total Volume / Surface Area

= (116-ft^3 / 0.40) / 110-ft^2 = **2.64-ft or approximately 2'-8"**

An overflow spillway or drain is then sized to convey any runoff that exceeds the design volume to the peak rate management facility.

Rain Gardens
(Example calculations shown for Lot #1; Refer to Worksheet 5A for additional calculations)

Surface Area:
Find the minimum surface area for each rain garden based on the maximum loading ratios.

Maximum impervious area to infiltration area loading ratio = 5:1 (3:1 for Karst areas)
Tributary impervious area = 1,000-ft^2
= 1,000-ft^2 / 5 = **200-ft^2**
= minimum surface area of rain garden per impervious loading ratio

Maximum total drainage area to infiltration area loading ratio = 8:1
Total drainage area = 6,000-ft^2 (typ.)
= 4,775-ft^2 / 8 = **597-ft^2**
= minimum surface area of rain garden per pervious loading ratio

The larger of the two calculated areas is the minimum surface area required for the facility.

Minimum Rain Garden Surface Area = **597-ft^2**

Depth:
Design guidelines, from the *PA BMP Manual*, for rain gardens limit ponding depth within the facility to 12 inches or less. The rain gardens in this example have been designed with a total ponding depth of 12 inches. The overflow outlets are positioned 6 inches above the bottom elevation of the rain gardens and 6 inches of freeboard is provided above the overflow outlets.

Volume:
The total detention volume of the rain garden is calculated by multiplying the surface area of the rain garden by the total depth. The 6 inches of water below the overflow outlet will be infiltrated and the remaining depth is used as short-term retention while flow is regulated through the overflow device. When calculating the infiltration volume, the bottom surface area of the BMP must be used.

Infiltration Volume = Surface Area x Depth
= 700-ft^2 x 0.5-ft = **350-ft^3**

Bioretention
(Refer to *Worksheet 5A* for additional calculations)

Surface Area:
Find the minimum surface area for the bioretention facility based on the maximum loading ratios.

Maximum impervious area to infiltration area loading ratio = 5:1 (3:1 for Karst areas)
Tributary impervious area = 9,700-ft^2 (typ.)
= 9,700-ft^2 / 5 = **1,940-ft^2**
= minimum surface area of Infiltration Trench per impervious loading ratio

Maximum total drainage area to infiltration area loading ratio = 8:1
Total drainage area = 41,400-ft^2
= 41,400-ft^2 / 8 = **5,175-ft^2**
= minimum surface area of Infiltration Trench per pervious loading ratio

The larger of the two calculated areas is the minimum surface area required for the facility.

Minimum Infiltration Trench Surface Area = **5,175-ft^2**

Depth:
The bioretention facility in this example has been designed with a total depth of 18 inches. The overflow outlets are positioned 6 inches above the bottom elevation, and 12 inches of freeboard is provided above the overflow outlets.

Volume:
The total detention volume of the bioretention facility is calculated by multiplying the surface area by the total depth. The 6 inches of water below the overflow outlet will be infiltrated and the remaining depth is used as short-term retention while flow is regulated through the overflow device. When calculating the infiltration volume, the bottom surface area of the BMP must be used.

Infiltration Volume = Surface Area x Depth
= 5,175-ft^2 x 0.5-ft = **2,487.5-ft^3**

STRUCTURAL CONTROL VOLUME REQUIREMENT CHECK

(Refer to *Worksheet 5*)

Check the total structural volume to be certain it is adequate to meet the structural volume requirement.

= Total Structural Volume - Structural Volume Requirement
=14,613-ft^3 – 11,984-ft^3 = 2,629-ft^3

The structural volume requirement has been exceeded by 2,629-ft^3 and no further BMP calculations are necessary.

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Engineering & Related Services

Project Name: DESIGN EXAMPLE 1
Project ID: MILL RUN RESIDENTIAL
Owner:
Calculated: Date:
Checked: Date:

WORKSHEET 1. GENERAL SITE INFORMATION

INSTRUCTIONS: Fill out *Worksheet 1* for each watershed

Date: 2/29/2009

Project Name: DESIGN EXAMPLE 1

Municipality: GRANVILLE TOWNSHIP

County: MIFFLIN

Total Area (acres): 9.78

Major River Basin: SUSQUEHANNA RIVER
http://www.dep.state.pa.us/dep/deputate/watermgt/wc/default.htm#newtopics

Watershed: JUNIATA RIVER

Sub-Basin: N/A

Nearest Surface Water(s) to Receive Runoff: MILL RUN

Chapter 93 - Designated Water Use: CWF
http://www.pacode.com/secure/data/025/chapter93/chap93toc.html

Impaired according to Chapter 303(d) List? http://www.dep.state.pa.us/dep/deputate/watermgt/wqp/wqstandards/303d-Report.htm	Yes	
	No	X
List Causes of Impairment:		

Is project subject to, or part of:

Municipal Separate Storm Sewer System (MS4) Requirements? http://www.dep.state.pa.us/dep/deputate/watermgt/wc/Subjects/StormwaterManagement/GeneralPermits/default.htm	Yes	
	No	X
Existing or planned drinking water supply?	Yes	
	No	X
If yes, distance from proposed discharge (miles):		
Approved Act 167 Plan? http://www.dep.state.pa.us/dep/deputate/watermgt/wc/Subjects/StormwaterManagement/Approved_1.html	Yes	X
	No	
Existing River Conservation Plan? http://www.dcnr.state.pa.us/brc/rivers/riversconservation/planningprojects/	Yes	
	No	X

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Project Name: DESIGN EXAMPLE 1
Project ID: MILL RUN RESIDENTIAL
Owner:
Calculated: Date:
Checked: Date:

WORKSHEET 2. SENSITIVE NATURAL RESOURCES

INSTRUCTIONS:

1. Provide Sensitive Resources Map according to non-structural BMP 5.4.1 in Chapter 5 of PA Stormwater BMP Manual. This map should identify waterbodies, floodplains, riparian areas, wetlands, woodlands, natural drainage ways, steep slopes, and other sensitive natural areas.

2. Summarize the existing extent of each sensitive resource in the Existing Sensitive Resources Table (below, using Acres). If none present, insert 0.

3. Summarize Total Protected Area as defined under BMPs in Chapter 5.

4. Do not count any area twice. For example, an area that is both a floodplain and a wetland may only be considered once.

EXISTING NATURAL SENSITIVE RESOURCE	MAPPED? yes/no/n/a	TOTAL AREA (Ac.)	PROTECTED AREA (Ac.)
Waterbodies	yes	0.00	
Floodplains	no	0.00	
Riparian Areas	no	0.00	
Wetlands	no	0.00	
Woodlands	yes	2.29	1.31
Natural Drainage Ways	N/A	0.00	
Steep Slopes, 15% - 25%	N/A	0.00	
Steep Slopes, over 25%	N/A	0.00	
Other:	N/A		
Other:	N/A		
TOTAL EXISTING:		**2.29**	**1.31**

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Project Name: DESIGN EXAMPLE 1
Project ID: MILL RUN RESIDENTIAL
Owner:
Calculated: Date:
Checked: Date:

WORKSHEET 3. NON-STRUCTURAL BMP CREDITS

PROTECTED AREA

1.1 Area of Protected Sensitive/Special Value Features (see WS 2)	1.31	Ac.
1.2 Area of Riparian Forest Buffer Protection	0.00	Ac.
3.1 Area of Minimum Disturbance/Reduced Grading	0.37	Ac.
TOTAL	1.68	Ac.

Site Area	*minus*	Protected Area	=	Stormwater Management Area
9.78	-	1.68	=	8.10

VOLUME CREDITS

3.1 Minimum Soil Compaction							
Lawn	16,165	ft^2	x 1/4"	x 1/12	=	337	ft^3
Meadow	N/A	ft^2	x 1/3"	x 1/12	=	0	ft^3
3.3 Protect Existing Trees							
For Trees within 100 feet of impervious area:							
Tree Canopy	N/A	ft^2	x 1/2"	x 1/12	=	0	ft^3
For Trees within 20 feet of impervious area:							
Tree Canopy	N/A	ft^2	x 1"	x 1/12	=	0	ft^3
5.1 Disconnect Roof Leaders to Vegetated Areas							
For runoff directed to areas protected under 5.8.1 and 5.8.2							
Roof Area	N/A	ft^2	x 1/3"	x 1/12	=	0	ft^3
For all other disconnected roof areas							
Roof Area	N/A	ft^2	x 1/4"	x 1/12	=	0	ft^3
5.2 Disconnect Non-Roof Impervious to Vegetated Areas							
For Runoff directed to areas protected under 5.8.1 and 5.8.2							
Impervious Area	10,000	ft^2	x 1/3"	x 1/12	=	278	ft^3
For all other disconnected non-roof impervious areas							
Impervious Area	N/A	ft^2	x 1/4"	x 1/12	=	0	ft^3
TOTAL NON-STRUCTURAL VOLUME CREDIT*						615	ft^3

* *For use on Worksheet 5*

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Project Name: DESIGN EXAMPLE 1
Project ID: MILL RUN RESIDENTIAL
Owner:
Calculated: Date:
Checked: Date:

WORKSHEET 4. CHANGE IN RUNOFF VOLUME FOR 2-YR STORM EVENT

PROJECT: DESIGN EXAMPLE 1
Drainage Area: 8.10 (acres)
2-Year Rainfall: 2.76 inches *(From NOAA Atlas 14)*

Total Site Area: 9.78 acres
Protected Site Area: 1.68 acres
Stormwater Management Area: 8.10 acres *(From Worksheet 3)*

Existing Conditions:

Land Use	Soil Type (HSG)	Area (sf)	Area (acres)	CN	S	Ia (0.2*S)	Q Runoff[1] (in)	Runoff Volume[2] (ft^3)
Woods (good)	B	42,500	0.98	55	8.1818	1.6364	0.14	481
Meadow	B	310,255	7.12	58	7.2414	1.4483	0.20	5,201
								-
								-
								-
								-
TOTAL:		**352,755**	**8.10**					**5,682**

Developed Conditions:

Land Use	Soil Type (HSG)	Area (sf)	Area (acres)	CN	S	Ia (0.2*S)	Q Runoff[1] (in)	Runoff Volume[2] (ft^3)
Meadow	B	54,060	1.24	58	7.2414	1.4483	0.20	906
Open Space (good)	B	243,035	5.58	61	6.3934	1.2787	0.28	5,643
Impervious	B	55,660	1.28	98	0.2041	0.0408	2.53	11,732
								-
								-
								-
TOTAL:		**352,755**	**8.10**					**18,281**

2-Year Volume Increase (ft^3):	12,599

2-Year Volume Increase = Developed Conditions Runoff Volume - Existing Conditions Runoff Volume
= 18,281 - 5,682 = 12,599 ft^3

1. Runoff (in) = Q = $(P - 0.2S)^2 / (P + 0.8S)$ where
   P = 2-Year Rainfall (in)
   S = (1000/ CN)-10

2. Runoff Volume (CF) = Q x Area x 1/12
   Q = Runoff (in)
   Area = Land use area (sq. ft)

Note: Runoff Volume must be calculated for EACH land use type/condition and HSG. The use of a weighted CN value for volume calculations is not acceptable.

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Project Name: DESIGN EXAMPLE 1
Project ID: MILL RUN RESIDENTIAL
Owner:
Calculated: Date:
Checked: Date:

WORKSHEET 5. STRUCTURAL BMP VOLUME CREDITS

SUB-BASIN: N/A

Check 25% Limit for Non-Structural BMP Credits: 615
÷ 12,599
4.9%

Required Control Volume (ft^3): 12,599
Allowable Non-structural Volume Credit (ft^3): - 615

Structural Volume Reqmt (ft^3): 11,984
(Required Control Volume minus Non-structural Credit)

Proposed BMP		Area (ft^2)	Infiltration Volume (ft^3)
6.4.1	Porous Pavement		
6.4.2	Infiltration Basin		
6.4.3	Infiltration Bed		
6.4.4	Infiltration Trench		
6.4.5	Rain Garden/Bioretention	11,915	8,827
6.4.6	Dry Well / Seepage Pit	4,400	5,787
6.4.7	Constructed Filter		
6.4.8	Vegetated Swale		
6.4.9	Vegetated Filter Strip		
6.4.10	Berm		
6.5.1	Vegetated Roof		
6.5.2	Capture and Re-use		
6.6.1	Constructed Wetlands		
6.6.2	Wet Pond / Retention Basin		
6.6.3	Dry Extended Detention Basin		
6.6.4	Water Quality Filters		
6.7.1	Riparian Buffer Restoration		
6.7.2	Landscape Restoration / Reforestation		
6.7.3	Soil Amendment		
6.8.1	Level Spreader		
6.8.2	Special Storage Areas		
Other			

Total Structural Volume (ft^3): 14,613
Structural Volume Requirement (ft^3): 11,984

DIFFERENCE: 2,629 *(excess)*

* Complete BMP Design Checklist for each measure proposed
NOTE: Provide supporting Volume Calculations for each Structural BMP

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Project Name: DESIGN EXAMPLE 1
Project ID: MILL RUN RESIDENTIAL
Owner:
Calculated: Date:
Checked: Date:

WORKSHEET 5.A - INFILTRATION BMP SUPPORTING CALCULATIONS

Instructions: Complete this worksheet for each Point of Interest / Discharge (at a minimum)

Point of Interest / Discharge: *Basin Outfall*
Total Drainage Area to POI: 352,755 ft²
Total Impervious Area: 55,660 ft²

	Proposed Infiltration BMP(s)	Infiltration Rate: Measured Infilt Rate[1] in/hr	Infiltration Rate: FOS	Infiltration Rate: Design Infilt Rate in/hr	Design Runoff Volume cu. ft.	Infiltration Period: Infilt. Period[2] hrs	Infiltration Period: Active Infilt.[3] hrs	Infiltration Period: Total Infilt Period hrs	Hydraulic Loading – Imperv. Drainage Area Loading: Imperv. DA sq. ft.	% area draining to BMP	Imperv Area Loading Ratio	Imperv Target Area sq. ft.	Hydraulic Loading – Total Drainage Area Loading: Total Drainage Area sq. ft.	% area draining to BMP[4]	Total Area Loading Ratio	Total Target Area sq. ft.	Target BMP Area[5] sq. ft.	Actual BMP Area[6] sq. ft.	Computed Infiltration Volume cu. ft.
BMP 6.4.1	Porous Pavement w. Infilt. Bed							Subtotal:			5				8				
BMP 6.4.2	Infiltration Basin							Subtotal:			5				8				
BMP 6.4.3	Subsurface Infiltration Bed							Subtotal:			5				8				
BMP 6.4.4	Infiltration Trench							Subtotal:			5				8				
BMP 6.4.5	Rain Garden/Bioretention							Subtotal:	19,700		5		89,595		8			11,915	5,827
	Rain Garden (Lot #1)	0.93	2	0.47	350	12.9	6	18.9	1,000	100.0		200	4,775	100.0		597	597	700	513
	Rain Garden (Lot #2)	0.95	2	0.48	350	12.6	6	18.6	1,000	100.0		200	5,600	100.0		700	700	700	516
	Rain Garden (Lot #3)	0.98	2	0.49	350	12.2	6	18.2	1,000	100.0		200	5,080	100.0		635	635	700	522
	Rain Garden (Lot #4)	1.01	2	0.51	300	11.9	6	17.9	1,000	100.0		200	3,625	100.0		453	453	600	462
	Rain Garden (Lot #5)	1.02	2	0.51	468	11.8	6	17.8	1,000	100.0		200	7,005	100.0		876	876	915	691
	Rain Garden (Lot #6)	1.08	2	0.54	350	11.1	6	17.1	1,000	100.0		200	5,310	100.0		664	664	700	539
	Rain Garden (Lot #7)	0.91	2	0.46	300	13.2	6	19.2	1,000	100.0		200	4,395	100.0		549	549	600	437
	Rain Garden (Lot #8)	0.99	2	0.50	238	12.1	6	18.1	1,000	100.0		200	3,000	100.0		375	375	475	355
	Rain Garden (Lot #9)	1.05	2	0.53	300	11.4	6	17.4	1,000	100.0		200	4,520	100.0		565	565	600	458
	Rain Garden (Lot #10)	1.03	2	0.52	375	11.7	6	17.7	1,000	100.0		200	4,975	100.0		622	622	750	568
	Bioretention 1	0.92	2	0.46	2588	13.0	6	19.0	9,700	100.0		1,940	41,400	100.0		5,175	5,175	5,175	3,778
BMP 6.4.6	Dry Well / Seepage Pit							Subtotal:	21,500		5		25,900		8			4,400	5,787
	Dry Well (Lot #1)	0.98	2	0.49	468	26.0	6	32.0	2,150	100.0		430	2,590	100.0		324	430	440	576
	Dry Well (Lot #2)	0.91	2	0.46	468	28.1	6	34.1	2,150	100.0		430	2,590	100.0		324	430	440	568
	Dry Well (Lot #3)	1.06	2	0.53	468	24.1	6	30.1	2,150	100.0		430	2,590	100.0		324	430	440	585
	Dry Well (Lot #4)	1.02	2	0.51	468	25.0	6	31.0	2,150	100.0		430	2,590	100.0		324	430	440	580
	Dry Well (Lot #5)	0.93	2	0.47	468	27.4	6	33.4	2,150	100.0		430	2,590	100.0		324	430	440	570
	Dry Well (Lot #6)	1.07	2	0.54	468	23.9	6	29.9	2,150	100.0		430	2,590	100.0		324	430	440	586
	Dry Well (Lot #7)	0.97	2	0.49	468	26.3	6	32.3	2,150	100.0		430	2,590	100.0		324	430	440	575
	Dry Well (Lot #8)	1.01	2	0.51	468	25.3	6	31.3	2,150	100.0		430	2,590	100.0		324	430	440	579
	Dry Well (Lot #9)	1.04	2	0.52	468	24.5	6	30.5	2,150	100.0		430	2,590	100.0		324	430	440	582
	Dry Well (Lot #10)	1.07	2	0.54	468	23.9	6	29.9	2,150	100.0		430	2,590	100.0		324	430	440	586
BMP 6.4.7	Constructed Filter										5				8				
BMP 6.4.8	Vegetated Swale										5				8				
BMP 6.4.9	Vegetated Filter Strip										5				8				
BMP 6.4.10	Infilt. Berm & Ret. Grading										5				8				
																		TOTAL:	14,613

[1] Assumes a soil testing procedure which finds hydraulic conductivity. (e.g. perc tests may also require a reduction factor)
[2] Time it takes for BMP to empty once it is full. (Minimum = 24 hrs, Maximum = 72 hours; Applicable to retention and detention facilities only.)
[3] Infiltration that occurs during the storm (before becoming full). Not to exceed 6 hours.
[4] A portion of the total area draining to BMP from non-pervious area may be diverted.
[5] Inherent in these calculations are the allowable loading ratios (5:1 and 8:1) from the BMP Manual. Higher loading ratios will need to be justified. In Karst Areas, the max. loading ratio should be 3:1.

PEAK RATE CONTROL ANALYSIS

According to the National Engineering Handbook (NRCS, 2008), the direct runoff for watersheds having more than one hydrologic soil-cover complex can be estimated in either of two ways. Runoff can be estimated for each complex and then weighted to get the watershed average. Alternatively, the CN values can be weighted, based on area, to obtain a single CN value to represent the entire drainage area. Then runoff is estimated with the single CN value. If the CN for the various hydrologic soil-cover complexes are close in value, both methods of weighting give similar results for runoff. However, if there exists a large difference in curve number value, the CN weighting method can provide drastically different results.

As described in the *National Engineering Handbook*, "the method of weighted runoff always gives the correct result (in terms of the given data), but it requires more work than the weighted CN method, especially when a watershed has many complexes. The method of weighted CN is easier to use with many complexes or with a series of storms. However, where differences in CN for a watershed are large, this method either under- or over-estimates runoff, depending on the size of the storm." This often occurs when impervious area exists in a subarea. When the relatively low curve number of lawn areas is combined with the high curve number of impervious areas, the weighted CN method will minimize the impact of the impervious surface and under-estimate the amount of runoff.

The spatial distribution of the different soil-cover complexes becomes the controlling factor in selection of the appropriate method. When different land uses behave as independent watershed the areas should be analyzed as separate drainage subareas. For example, when a large parking area is surrounded by lawn area that all flows to the same collection point, runoff from the impervious surface will occur much differently than runoff from the lawn. However, when impervious area is dispersed amongst other land uses and not directly connected to a stormwater collection system, the weighted CN method may be appropriate. The decision of whether or not to use a weighted curve number is often a site specific judgment that should be discussed between the designer and the Municipal Engineer in the early planning stages of a project.

Pre-Development Soil-Cover Complex Data

Because the wooded area along the north property line will remain unchanged, and will not be tributary to the stormwater facilities, this area has been removed from the peak rate analysis drainage areas. The weighted CN method was used for pre-development calculations in this example because Curve Numbers for the hydrologic soil-cover complexes are close in value. The drainage area and land cover information necessary to calculate the pre-development runoff is shown in the table below:

Land Use	Soil Type (HSG)	Area (ft^2)	Area (acres)	CN
Woods (good)	B	42,500	0.98	55
Meadow	B	310,255	7.12	58
TOTAL:		352,755	8.10	58

Pre-Development Time of Concentration

The *Model Ordinance* requires use of the NRCS Lag Equation for all pre-development time of concentration calculations unless another method is pre-approved by the Municipal Engineer.

$$T_{lag} = L^{0.8}\frac{(S+1)^{0.7}}{1900\sqrt{Y}}$$

Where:

T_{lag} = Lag time (hours)
L = Hydraulic length of the watershed (feet)
Y = Average overland slope of watershed (percent)
S = Maximum retention in the watershed, as defined by: S = [(1000/CN) – 10]
CN = NRCS Curve Number for the watershed

Lag time is related to time of concentration by the following equation:

Time of Concentration = T_c = [(T_{lag}/.6) * 60] (minutes)

One method of calculating the average overland slope of a watershed is to select locations that represent the various slopes found in the watershed and weight the slope based on the area it represents. This method is shown in the table on the following page.

Slope	End Elevation		Distance	Slope	Percent of	Product
Line	High	Low	(ft)	(%)	Total Area	(% x %)
AA	909	902	148	4.7%	5%	0.24%
BB	941	909	475	6.7%	50%	3.37%
CC	956	942	245	5.7%	15%	0.86%
DD	960	943	180	9.4%	15%	1.42%
EE	943	930	265	4.9%	15%	0.74%
					Sum of Products =	6.61%

This is an estimation of the land slope value, so the calculated number is rounded to the nearest whole number for use in the Lag Equation. The hydraulic length of the watershed was measured at 1050 ft. Therefore,

$$T_{lag} = (1050)^{0.8}\frac{((1000/CN)-10)+1)^{0.7}}{1900\sqrt{7}}$$

T_{lag} = 0.23 hours

Time of Concentration = T_C = (T_{lag} / 0.6) * 60
= (0.23 / 0.6) * 60
= 23 minutes

Pre-Development Peak Rate Flows

All of this information was used to perform a pre-development peak rate analysis using a software package based on the NRCS TR-20 procedures. The results of the analysis are as follows:

	1-year	2-year	10-year	25-year	50-year	100-year
Peak Runoff Flows (cfs)	0.1	0.6	4.1	7.6	11.1	15.3
Runoff Volume (ac-ft)	0.060	0.136	0.449	0.726	0.997	1.322
Runoff Depth (in)	0.09	0.20	0.66	1.08	1.48	1.96

Table B.1. Pre-Development Runoff Summary

Post-Development Soil-Cover Complex Data
Due to the disconnection of impervious areas and overland flow paths used in this design, the area weighted CN method was deemed appropriate and used to reduce the complexity of the model. The drainage area and land cover information for the drainage sub-area directly tributary to the bioretention facility is shown in the table below:

Land Use	**Soil Type (HSG)**	**Area (ft^2)**	**Area (acres)**	**CN**
Lawn (good condition)	B	9,700	0.22	61
Impervious	B	31,700	0.73	98
TOTAL:		41,400	0.95	70

Post-Development Time of Concentration
The Segmental Method was used for all post-development time of concentration calculations in this example. This method is covered in more detail in various NRCS publications (NRCS, 1986; NRCS, 2008). The following segments were used to calculate a time of concentration for the drainage sub-area directly tributary to the bioretention facility:

T_t*-1*: Sheet flow, 100' of lawn at 5% = 10.7 min
T_t*-2*: Shallow concentrated flow, 110' unpaved at 5.9% = 0.5 min
T_t*-3*: Channel flow, 80' at 4.0% = 0.2 min
T_t*-4*: Channel flow, 156' at 3.85% = 0.5 min
T_t*-5*: Pipe flow, 38' of 15" HDPE pipe at 5.2% = 0.1 min

$T_c = T_t\text{-}1 + T_t\text{-}2 + T_t\text{-}3 + T_t\text{-}4 + T_t\text{-}5$ = 12 minutes

Post-Development Peak Rate Flows
The hydrologic model for this example contains a considerable level of detail. Each structural BMP was modeled as a pond with a unique drainage area and time of concentration. Runoff was routed through each BMP and linked to downstream BMPs for subsequent routing. A detention basin with an outlet control structure was also added to the model. A graphical representation of the model is provided in *Figure B.1*.

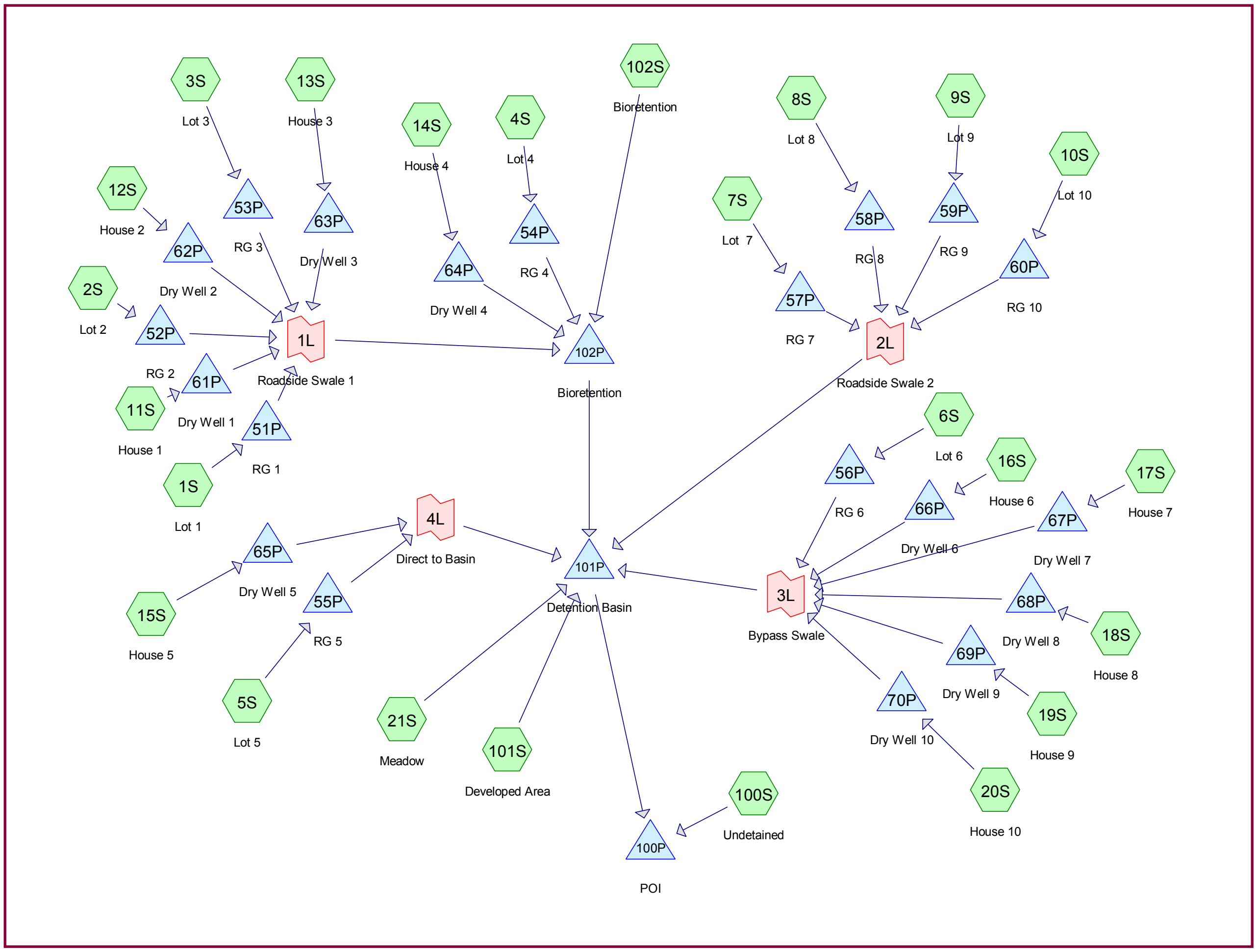

Figure B.1. Hydrologic Model of Post-Development Conditions

This model was used to estimate the post-development peak rate flows. The final configuration of the outlet structure was completed through an iterative process using the results of the model runs. This design meets the peak rate control requirements through a combination of volume removed by the structural `BMPs and the detention basin and outlet control structure. *Table B.2* shows a summary of the runoff results for the final post-development design:

	1-year	2-year	10-year	25-year	50-year	100-year
Peak Runoff Flows (cfs)	0.1	0.4	4.1	7.4	10.6	15.2
Runoff Volume (ac-ft)	0.079	0.147	0.445	0.717	1.011	1.367
Runoff Depth (in)	0.12	0.22	0.66	1.06	1.50	2.03

Table B.2. Summary of Post-Development Runoff with Stormwater Controls

INITIAL CONSTRUCTION COST - DESIGN EXAMPLE

Initial construction costs were estimated for each layout. The estimates include the costs incurred by the developer to complete earthwork, paving and curbing, and stormwater management facilities. All of these costs are summed to determine an initial construction cost for these facilities. This cost was then divided by the total sellable acreage of the project to determine a cost / sellable acre for each layout.

Estimate of Initial Construction Cost
Mill Run Residential – Traditional Layout

ITEM NO.	ITEM & DESCRIPTION	EST.	UNIT	UNIT PRICE	EXTENSION
EARTHWORK				**Subtotal =**	**$ 23,950**
1	Clearing & Grubbing	2.3	AC	$ 6,000.00	$ 13,800
2	Topsoil Removal/Stockpiling	5.8	AC	$ 1,750.00	$ 10,150
STORM DRAINAGE				**Subtotal =**	**$ 102,769**
3	Storm Sewer, 18" HDPE	600	LF	$ 55.00	$ 33,000
4	Storm Inlets	7	EA	$ 2,100.00	$ 14,700
5	Swales	490	LF	$ 10.00	$ 4,900
6	Install Detention Basin	1,525	CY	$ 25.00	$ 38,125
7	Anti Seep Collars	2	EA	$ 775.00	$ 1,550
8	Outlet Structure	1	EA	$ 4,000.00	$ 4,000
9	Outlet Pipe, 18" HDPE	50	LF	$ 55.00	$ 2,750
10	DW Endwall 24"	1	EA	$ 2,750.00	$ 2,750
11	Rip Rap Apron	144	SF	$ 6.90	$ 994
PAVING & CURBING				**Subtotal =**	**$ 138,657**
12	Paving - Final Subgrade, 6" Stone, 3" 19MM, 1-1/2" 9.5mm	2,325	SY	$ 30.00	$ 69,750
13	Curbing w/Excavation & Backfill	1,465	LF	$ 27.00	$ 39,555
14	Sidewalk plain w/4" - stone	4,285	SF	$ 6.85	$ 29,352
				Initial Construction Cost =	**$ 265,376**
				Cost / Sellable Acre =	**$ 42,734**

Table B.3. Estimate of Construction Cost for Residential Design Example (Traditional Layout)

ITEM NO.	ITEM & DESCRIPTION	EST.	UNIT	UNIT PRICE	EXTENSION
Estimate of Initial Construction Cost *Mill Run Residential – LID Layout*					
EARTHWORK				**Subtotal =**	**$ 14,925**
1	Clearing & Grubbing	1.0	AC	$ 6,000.00	$ 6,000
2	Topsoil Removal/Stockpiling	5.1	AC	$ 1,750.00	$ 8,925
STORM DRAINAGE				**Subtotal =**	**$ 114,172**
3	Swales	1,620	LF	$ 10.00	$ 16,200
4	Storm Sewer, 18" HDPE	136	LF	$ 55.00	$ 7,480
5	DW Headwall 18"	1	EA	$ 2,750.00	$ 2,750
6	Storm Inlets	1	EA	$ 2,100.00	$ 2,100
7	Install Detention Basin	600	CY	$ 25.00	$ 15,000
8	Anti Seep Collars	2	EA	$ 775.00	$ 1,550
9	Outlet Structure	1	EA	$ 4,000.00	$ 4,000
10	Outlet Pipe, 18" HDPE	50	LF	$ 55.00	$ 2,750
11	Level Spreader	44	LF	$ 5.50	$ 242
12	Bioretention Area	5,175	SF	$ 12.00	$ 62,100
PAVING & CURBING				**Subtotal =**	**$ 53,790**
13	Paving - Final Subgrade, 6" Stone, 3" 19MM, 1-1/2" 9.5mm	1,645	SY	$ 30.00	$ 49,350
14	Gravel Shoulder	370	SY	$ 12.00	$ 4,440
				Initial Construction Cost =	**$ 182,887**
				Cost / Sellable Acre =	**$ 28,355**

Table B.4. Estimate of Construction Cost for Residential Design Example (LID Layout)

The cost of constructing the stormwater BMPs on each individual lot was not included in the comparison of initial construction costs. This is a cost that will be borne by the owner of each individual lot. This must be included in the cost comparison analysis. *Table B.5* shows an estimate of these costs.

ITEM NO.	ITEM & DESCRIPTION	EST.	UNIT	UNIT PRICE	EXTENSION
Estimate of Stormwater BMP Construction Cost *Mill Run Residential – LID Layout*					
STORMWATER BMPS					
1	Rain Gardens	6,740	SF	$ 10.00	$ 67,400
2	Dry Wells	450	CY	$ 32.00	$ 14,400
				Construction Cost =	**$ 81,800**
				Cost / Sellable Acre =	**$ 12,682**

Table B.5. Estimate of Stormwater BMP Construction Cost

Determining how this additional cost to homeowners will be reflected in the market value of developed land is presumptive at best. For this example, we have assumed that some of the cost of constructing the stormwater BMPs will result in a dollar for dollar reduction in the market value of the sellable land. So, the BMP construction cost per sellable acre is subtracted from the per acre market value price of the land.

The initial construction cost is subtracted from the land sale value to determine the developers profit for each layout.

Cost = Land Sale Value - Initial Construction Cost

Traditional Layout
Cost = $310,500 - $265,376
= $45,124

LID Layout
Cost = $240,701 – $182,887
= $57,814

The final cost comparison is completed by determining the difference in profit between the two layouts. For this example, a total profit increase of $12,690 is realized by the developer using the LID layout with no additional cost to the individual homeowners.