STORMWATER MANAGEMENT PLAN
FOR
THE SPRING CREEK WATERSHED
Centre County, Pennsylvania

June 21, 2002

AN INITIATIVE OF THE SPRING CREEK WATERSHED COMMISSION
THROUGH
THE CENTRE COUNTY BOARD OF COMMISSIONERS

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IN COOPERATION WITH
THE CENTRE COUNTY PLANNING OFFICE

EXECUTIVE SUMMARY

On behalf of the Spring Creek Watershed Commission, the Centre County Commissioners, through the Centre County Planning Office, have prepared a Stormwater Management Plan for the Spring Creek Watershed to comply with the Stormwater Management Act of 1978 (Act 167). The goal of the Spring Creek Watershed Stormwater Management Plan is to provide all municipalities within the watershed a technically sound and administratively consistent implementation procedure for comprehensive stormwater quantity and quality management. Since runoff does not recognize municipal boundaries, this plan will provide a consistent watershed-wide approach to stormwater management.

In addition to the overall goal identified above, a specific technical strategy to manage stormwater discharges from new land development has been developed for the Watershed. This strategy includes consideration of the unique hydrologic processes that occur within the drainage basin due to the carbonate geology underlying the majority of the Watershed. The technical strategy presented in this plan meets the following performance criteria:

- Provide improved control of increased runoff from land development activities;
- Manage overbank and extreme flood events;
- Maintain groundwater recharge;
- Reduce channel erosion;
- Minimize non-point source pollution resulting from urban runoff through implementation of pollutant removal methodologies (Best Management Practices);
- Preserve and protect well-head areas; and
- Encourage low-impact development.

The Stormwater Management Plan includes a description of watershed characteristics, a presentation of technical standards, water quality considerations, an economic analysis, and recommendations for plan adoption and implementation. A model Stormwater Management Ordinance and a detailed technical analysis supporting the recommended technical standards are included as appendixes to the document.

Watershed Characteristics and Data Summaries

The following watershed characteristics and data summaries are included in the plan:

- Existing and future land uses;
- Soils;
- Geology and significant karst features;
- Hydrologic features;
- Inventory of significant flow obstructions;
- Inventory of stormwater related problem areas;
- Existing stormwater management facilities;
- Stream flow and precipitation data;
- Floodplain information;
- Water quality data; and
- Existing engineering and planning.

Most of the watershed characteristics and data summaries were also compiled within a Geographic Information System (GIS) database for future reference. Where appropriate this information is also presented graphically in the report.

Technical Standards

The following technical standards and criteria related to stormwater quantity and quality control are included in the Plan and apply to all land development activities within the Watershed.

Exemptions

The following land development activities are exempt from the requirement to submit a formal stormwater management plan and narrative as a part of the municipal land development review process:

- Development activities resulting in minor increases in impervious coverage;

- Land disturbance associated with the construction or alteration of one- and two-family dwellings;
- Any site less than one (1) acre in size that decreases the total site impervious area following development (with exceptions).

These exemptions do not relieve the applicant from the responsibility to implement on-site drainage control measures as necessary to protect the health, safety, and property of downstream residents. These measures include adequate and safe conveyance of stormwater on the site and as it leaves the site. In addition, these exemptions do not relieve the applicant from meeting requirements for water quality and ground water recharge, including special requirements for high quality (HQ) and exceptional value (EV) watersheds (DEP Chapter 93 and anti-degradation requirements), and water quality and recharge volume requirements.

Special Management Areas and Developments

Special management areas and water quality sensitive developments have been identified that require special consideration with regard to stormwater management. **Special management areas (or sensitive areas)** are defined as those areas that, if developed, have the potential to cause catastrophic loss to a water system well field. **Water quality sensitive (WQS) developments** are defined as land development projects that have a high potential to cause catastrophic loss to local water quality and could potentially threaten ground water reservoirs. WQS developments include vehicle fueling stations, industrial manufacturing sites, salvage yards, recycling centers, hazardous material storage areas, and interstate highways.

Standards and Calculation Methodologies

Standards and calculation methodologies have been developed to simplify stormwater management designs, unify methods, remove model parameter subjectivity, remove improperly used methods, and ensure that stormwater management decisions are based more realistically on hydrologic processes. New standards and criteria are established for stormwater quantity control, quality control, and conveyance.

Stormwater Quantity Control

Standards are established for limiting post-development runoff rates from regulated activities. Although these standards are similar to existing stormwater management requirements, there are many significant differences. Some of the more significant changes are as follows:

- Specific models and computational procedures are specified, including limitations on methods used for the time of concentration evaluation.
- New design storm frequencies, precipitation depths, rainfall distributions, and other design parameters are established.
- New criteria are presented for modeling undeveloped conditions.
- New procedures and criteria for modeling in karst and non-karst environments are presented.
- Additional considerations are provided for areas draining to closed depressions and sinkholes.
- Allowable minimum discharges from development sites are established.

In addition to providing new standards and computational methodologies for standard pre- and post-development runoff analysis, the following additional evaluation criteria are established:

- **Pond Capture Volume:** To minimize nuisance flooding from small precipitation events, a runoff capture volume will be required for all stormwater management ponds that do not discharge directly to natural well defined channels.
- **Recharge Volume:** To control runoff volume and minimize the loss of stream baseflow, a recharge volume criteria is established that must be evaluated for all development sites. This additional runoff volume will need to be controlled in a recharge practice. Recharge volume credits are for the use of low-impact development techniques such as sumping roof drains, directing sheet flow to filter / buffer strips, and maintaining existing site closed depressions. Due to the karst nature of the watershed, limitations have been placed upon the design of recharge practices.

Stormwater Quality Control

Water quality is implemented through a water quality volume (WQ_v) that must be evaluated for all development sites. Separate criteria are provided for sensitive and non-sensitive areas

within the Watershed, as well as for areas underlain by carbonate formations and non-carbonate formations. The water quality volume must be captured and filtered through a water quality BMP.

Three (3) types of credits can be applied to reduce the water quality volume to be treated. These credits, outlined below, are created as incentives to use environmentally sensitive design and low impact development planning techniques to minimize some of the adverse environmental impacts of land development activities.

- **Drainage-way Protection:** A water quality credit is given for preservation and protection of natural drainage-ways within the watershed.
- **Natural Area Conservation:** A water quality credit is given for natural areas that are conserved on the development site.
- **Filter / Buffer Areas:** A water quality credit is given when stormwater runoff is effectively treated via a vegetated filter / buffer area. Effective treatment is achieved when impervious area runoff is directed as sheet flow across vegetative filter strips.

Stormwater Conveyance

Standards are provided for analysis of stormwater conveyance in storm sewer piping systems, swales, and open channels. Although these standards are similar to existing stormwater management requirements, there are some differences. The most notable change is related to the recommended computational methods required for various drainage area sizes.

Water Quality Considerations

Non-point source (NPS) pollution is among the most significant environmental issues associated with urbanization and land development. Developed land areas (particularly impervious surfaces) accumulate a variety of urban pollutants from sources including atmospheric deposition, transportation activities, and lawn care and maintenance. These pollutants are washed off of developed surface areas during the initial flush of runoff from rainfall events and are delivered to downstream waters, potentially impacting aquatic life through sedimentation and chemical contamination.

To minimize adverse impacts to stream health resulting from stormwater NPS pollution, performance standards are provided for the implementation of Water Quality Best Management Practices (BMPs). Water quality BMPs are presented that include structural and non-structural techniques and devices to reduce or prevent the transport of NPS pollutants to receiving creeks, streams, and other bodies of water. Structural BMPs fall into the following five categories:

1. Stormwater Ponds;
2. Stormwater Wetlands;
3. Recharge Practices;
4. Filtering Practices;
5. Open Channel Practices.

Non-structural techniques include but are not limited to:

- Natural area conservation (including conservation of forested riparian buffers);
- Disconnection of rooftop and other impervious areas;
- Sheet flow discharge to stream buffers and filter strips;
- Use of grass lined open channels for conveyance;
- Environmentally sensitive or low impact rural development; and
- Impervious cover reduction.

A process for selecting an appropriate BMP or group of BMPs for a development site is also presented considering:

- Stormwater treatment suitability;
- Terrain factors;
- Physical feasibility factors; and
- Community and environmental factors.

Economic Analysis

An economic analysis was conducted to assess the actual cost of implementing the proposed stormwater management standards and criteria proposed. Ten (10) previously approved and constructed land development project sites within the Spring Creek Basin were re-analyzed using the proposed standards.

The test sites ranged in size from small commercial developments less than one acre in size to a site that included a drainage area of approximately 230 acres. The site land uses included commercial, institutional, light industrial, and single family and high-density residential developments. These project sites are located within five (5) different municipalities. This well mixed group of developments had impervious areas ranging from 16 to 67 percent.

Although the results of the analysis varied, it is concluded that implementation of the proposed standards will result in an average 15 to 20 percent increase in the cost of stormwater infrastructure for these sites. Having said this, it is recognized that actual costs may vary significantly (from no increase to a significant increase) depending on specific site conditions and the ability of the design professional (with the owner's cooperation) to develop innovative approaches to stormwater quantity and quality control. Design approaches utilizing non-structural BMPs will result in the lowest costs and may, in fact, actually reduce development costs in some instances.

Recommendations for Plan Adoption and Implementation

The Spring Creek Watershed Stormwater Management Plan preparation process is complete with the Centre County Commissioners' adoption of the Plan and submission of the Plan to PA DEP for approval. Subsequent activities necessary to carry out the provisions of the Plan are considered by DEP to be part of the implementation of the Plan.

The initial step in Plan implementation is DEP approval. Plan approval sets in motion the mandatory schedule of adoption of municipal ordinance provisions to implement the stormwater management standards and criteria included in the Plan. The municipalities within the Spring Creek Watershed will have six (6) months from the date of DEP approval in which to adopt the

necessary ordinance provisions. Failure to do so could result in the withholding of all state funds to the municipality in accordance with Act 167.

Other issues related to implementing a plan of this magnitude within the 14 municipalities located in the Watershed relate to consistent application of technical standards and maintenance of facilities. Creation of a single Stormwater Management Authority is a viable option to alleviate these problems and concerns and unify the implementation and monitoring of the Plan. However, creating an authority would not be a simple task and would require the cooperation of the municipalities involved in addition to the community at large.

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Chapter 1
INTRODUCTION

On behalf of the Spring Creek Watershed Commission, the Centre County Commissioners, through the Centre County Planning Office, have prepared this document to comply with the Stormwater Management Act of 1978 (Act 167). Act 167 requires that each county in Pennsylvania prepare and adopt Stormwater Management Plans for each designated watershed within the county. The Spring Creek Watershed is located entirely within Centre County. Figure 1-1 identifies the location of the watershed.

Recent and proposed future development activities in the Centre Region have and will continue to increase the potential for environmental degradation within the Spring Creek Watershed if measures to minimize impacts are not implemented. Significant land development and land use changes are occurring within the watershed as a result of economic growth and the development of an expanded interstate highway network that is greatly increasing the region's accessibility. The karst nature of the watershed adds an additional level of significance to the Plan. Not only will increased development have an impact on the high quality surface waters within the watershed, the carbonate geology creates direct connections between surface and ground waters, putting the region's high quality ground water at risk also. As a result, stormwater management, as it relates to both water quantity and quality, has been identified as the most important issue facing the watershed.

Act 167 – Background and Requirements

The Pennsylvania General Assembly passed the Stormwater Management Act (Act 167) and its companion bill, the Flood Plain Management Act (Act 166), on October 4, 1978. This legislation was enacted in response to increasing problems caused by inadequately controlled stormwater runoff and the associated threats to public health and safety. Acts 166 and 167 require a comprehensive approach to planning and managing excess stormwater runoff. Specifically, Act 167 establishes a program for managing accelerated runoff to avoid increased flooding, and Act

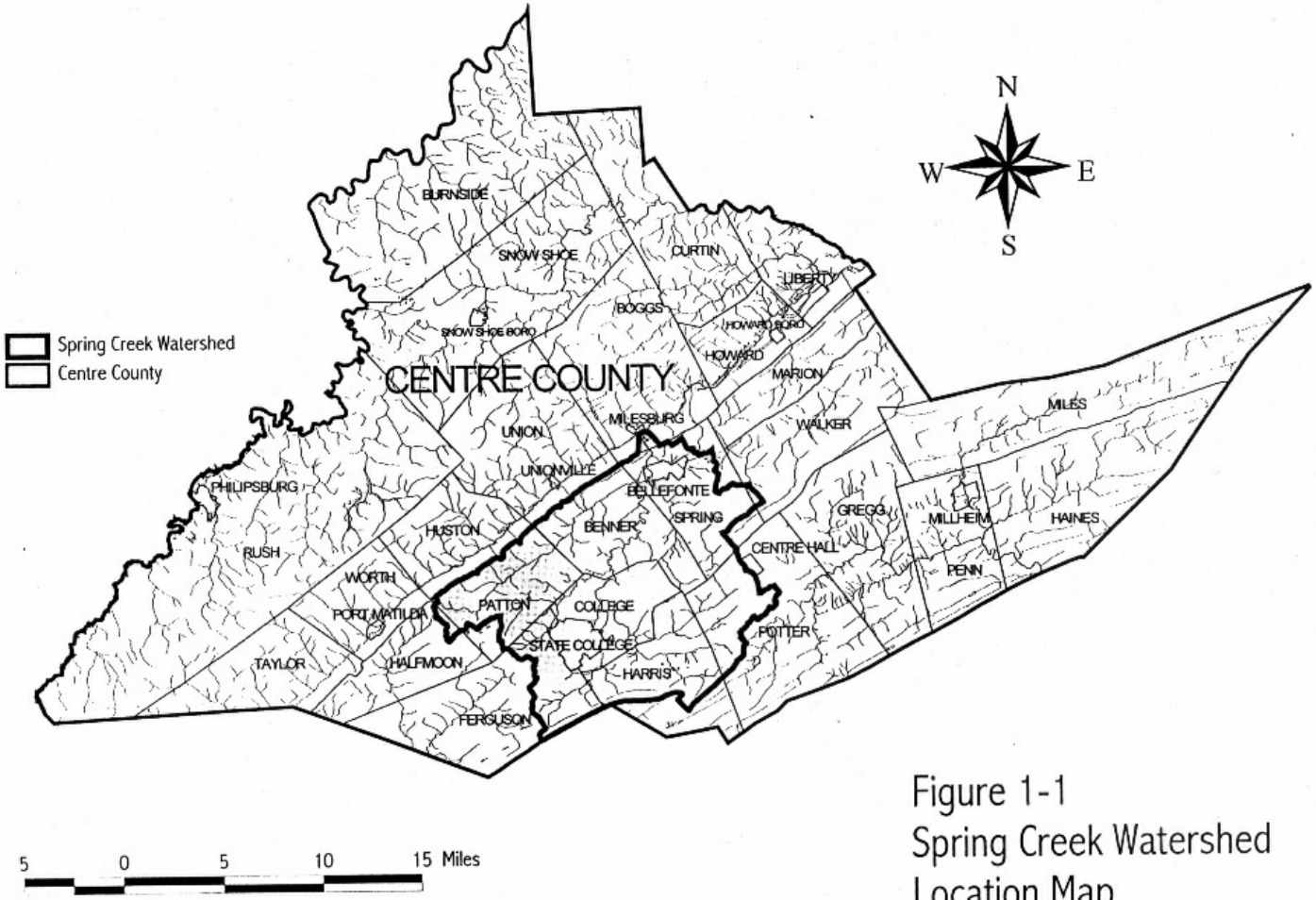

Figure 1-1
Spring Creek Watershed
Location Map

166 provides for the preservation and restoration of flood plains, that function as natural stormwater storage areas.

The following summary includes the basic elements of Act 167 in terms of specific responsibilities assigned to various units of state and local government:

1. Each county shall develop regional Stormwater Management Plans for each watershed within its boundaries, recognizing that many watersheds will cross county boundaries and will require collaboration with neighboring counties.

2. Each municipality within a designated watershed will adopt local ordinances and engineering design criteria that conform to the provisions of the respective Stormwater Management Plans.

3. Developers must implement stormwater management techniques that meet the standards and criteria set forth in the appropriate municipal ordinances, as supported by the watershed's Stormwater Management Plan. In general, these stormwater management techniques will ensure that post-development runoff rates throughout the watershed do not exceed pre-development levels.

4. PA DEP will serve as the review agency for each watershed's Stormwater Management Plan submitted by the counties. The Act 167 planning process involves three essential steps:

    a. Documentation of existing watershed characteristics, including land use, soils, runoff conditions, peak flows, sub-area timing relationships, existing storm drainage problems, and flow obstructions. The existing conditions in the watershed represent the baseline for evaluating the effects of future runoff caused by land development.

    b. Preparation of a watershed Stormwater Management Plan to manage stormwater based on possible future development and land use changes within the watershed.

The plan would include criteria and performance standards for managing urban runoff and a listing of alternative stormwater management techniques.

c. Development of priorities for implementing stormwater management practices within each municipality in accordance with the objectives set forth in the watershed Stormwater Management Plan. This step is crucial to the entire planning process because local level control is the only mechanism by which total watershed-wide stormwater management can be achieved. While this may seem contradictory to the objective of watershed-wide planning, Act 167 places the responsibility for managing excessive stormwater on each municipality.

Goals and Objectives

The goal of the Spring Creek Watershed Stormwater Management Plan is to provide all municipalities within the watershed a technically sound and administratively consistent implementation procedure for comprehensive stormwater management. Accordingly, the purpose of this Plan is to provide a watershed-wide approach to stormwater management because runoff does not recognize municipal boundaries.

In addition to the overall goal identified above, a specific technical strategy to manage stormwater discharges from new land development has been developed for the watershed. This strategy includes consideration of the unique hydrologic processes that occur within the drainage basin due to the carbonate geology underlying the majority of the watershed. Additionally, this Plan takes into account research on the problems associated with the use of hydrologic modeling practices related to nuisance flooding even when stormwater management practices are employed. The technical strategy presented in this Plan meets the following performance criteria:

- Minimize non-point source pollution resulting from urban runoff through implementation of pollutant removal methodologies (Best Management Practices);
- Maintain groundwater recharge;
- Reduce channel erosion;

- Reduce nuisance flooding resulting from application of inappropriate stormwater management techniques and procedures; and
- Manage overbank and extreme flood events.

The Model Ordinance, that is part of the Act 167 Plan, has been developed considering the unique hydrologic processes that occur within the Spring Creek Drainage Basin and how these processes can best be evaluated through the use of methods and procedures commonly used in engineering practice. This is not to say that the Model Ordinance, or the recommended analysis procedures, accurately simulate the physical hydrologic processes that occur in the Watershed. Rather, the proposed methodologies are designed to produce post-development stormwater designs that more closely mimic the existing hydrologic responses from pre-development conditions.

It is important to understand that it is not the intent of this Stormwater Management Plan to control or reduce development within the Watershed. However, the Plan does provide standards and criteria that can be incorporated into local ordinances to manage stormwater quantity (peak flows) and quality throughout the Watershed as development occurs. Also, it is not the intent of this plan to solve existing flooding or runoff problems, but to identify them for future correction and assure that problems do not get worse. More specifically, this Plan does not require that municipalities correct existing drainage problems.

It is also important to recognize that Act 167 only provides authority for the development of stormwater management standards and procedure. Although zoning and land-use planning can be critical to successful stormwater management, Act 167 does not provide the authority to modify zoning and land use regulations. Therefore, this document focuses only on technical standards and procedures for stormwater quantity and quality control.

Special Considerations

The Spring Creek Basin is mainly underlain by carbonate geology where the soil infiltration capacity is rarely limiting. As a result, most natural land areas do not produce overland or surface runoff. These areas contribute to the storm flow component of runoff via a greater than normal interflow component. Although this karst influence is the dominant hydrologic influence

in the Watershed, there are distinct areas and times when this is not true. Major floods in the basin typically result from snowmelt runoff events that occur when the ground is frozen or the upper soil layers are saturated. There are also areas where the processes of both infiltration excess and saturation excess dominate the generation of surface runoff. Additionally, there are areas that exfiltrate groundwater due to a combination of local soils and the underlying bedrock. Although the Model Ordinance has been developed attempting to account for all of these influences from a stormwater management perspective, stormwater management designs that ignore these special hydrologic conditions will usually cause unforeseen stormwater related problems. Therefore, where warranted, municipalities and reviewers must allow for unique analysis procedures and design as long as they can be shown to accurately reflect actual hydrological conditions occurring at the site.

In reviewing the current practices used in stormwater management modeling methods in the drainage basin, several major areas of concern were identified. An attempt was made to address each of these problems in the Model Ordinance using a consistent and theoretically correct procedure.

The first area of concern within the drainage basin (that is amplified due to the carbonate geology) is the generation and concentration of surface point discharges from developed areas. Often these same land areas, prior to being developed, never produced visible surface runoff except during snowmelt events under frozen ground conditions. The change in the hydrologic response of these land areas following development is one of the major causes of nuisance flooding that is reported below newly constructed stormwater management facilities. This is commonly a result of over-estimating the pre-development runoff rate for low frequency events. However, nuisance flooding also results from the use of design event peak runoff rates to design stormwater management facilities. Even when a 1-year design event is used for a stormwater management pond's design, a radical change in the frequency, volume, and runoff rates results. This is a direct result of the increase in runoff produced by impervious surfaces for precipitation events that previously caused no surface runoff. This additional runoff volume can become a significant source of downstream drainage problems.

A second problem that has been observed within the basin is the unrealistically low estimation of peak runoff rates for low frequency events (50- and 100-year). This problem often goes

unnoticed because of the infrequency of these extreme events. However, several stormwater management facilities within the basin have already experienced higher water surface elevations than their designed 100-year elevations, even though no such precipitation or snowmelt events have occurred. The problem has occurred as a result of engineers and municipalities incorporating model corrections that produce unrealistically low estimates for the low frequency events while trying to correct the nuisance flooding that occurs during more frequent events. The net result is that stormwater management facilities can be inadequately designed to safely pass true 50- or 100-year runoff events.

A third area of concern is the current lack of urban stormwater quality controls within the Basin. Because of the predominant carbonate geology within the Spring Creek Watershed, there is a real potential for ground water contamination from urban stormwater runoff. Therefore, water quality, capture depths, and recharge requirements have been incorporated into the Plan to protect water supplies and maintain baseflow within the Basin.

The Model Ordinance has also been developed to implement consistent computational methods within the Basin. One example of this is in the selection of precipitation events used in hydrologic models. The basin is located at a point on the Pennsylvania Department of Transportation (PA DOT) Precipitation Intensity, Duration, and Frequency (IDF) maps that indicate that half of the basin is in Region 2 and half of the basin is in Region 3. Unfortunately, very few people understand the statistical methods or relative probabilities that were used in developing the IDF map, or the map's relative accuracy. As a result, it is unrealistic to assume that one neighborhood lies within Region 2 while another lies in Region 3, especially when the entire watershed experiences similar precipitation events. In addition, hydrologic modeling methods in current stormwater management ordinances encourage the use of inconsistent computational methodologies. Therefore, the technical standards proposed herein place limits on which hydrologic methods may be used for stormwater management analysis within the Watershed.

Finally, the evaluation and interpretation of current stormwater management ordinances is often overly rigid. Municipal engineers and reviewers often require that design computations be shown to an accuracy of 1/100 of a cfs without giving consideration to the accuracy of the

computational methodologies being applied. It is important that computational accuracy and common sense be applied in the interpretation and evaluation of stormwater plans.

Project Coordination and Oversight

Implementation of a watershed-wide Stormwater Management Plan requires involvement and input from all stakeholders. For this Plan, stakeholders include numerous municipalities, various state and federal agencies, as well as special interest groups within the Watershed. To initiate municipal involvement, the Centre County Commissioners established the Spring Creek Watershed Commission, an organization comprising one elected municipal official from each municipality within the Spring Creek Watershed. The Spring Creek Watershed Commission acts as an intermunicipal organization for all activities within the Spring Creek Watershed.

In addition, Act 167 allows municipalities, conservation districts, and other interest groups to provide input and direction during the planning process through participation in a Watershed Plan Advisory Committee (WPAC). Individuals involved in the WPAC are listed on the next page.

In addition, a Technical Advisory Committee (TAC) was formed to offer technical expertise. Each municipality within the Watershed was solicited to appoint a representative to the TAC. The TAC also included other members of the community with expertise related to stormwater management. A list of people involved in the TAC also follows.

Both the WPAC and TAC promoted municipal and stakeholder involvement that ensured inter-municipal and other agency cooperation. These groups ultimately aided in the overall preparation of the plan. Their involvement is key to the Plan's implementation, long-term success, and effectiveness.

WPAC Member Affiliation

Mr. Roy Herlocker, Jr.	Bellefonte Borough
Mr. Richard Lahr	Benner Township
Ms. Xochi Confer	Boggs Township
Mr. Robert Sweitzer	Centre County Conservation District
Mr. Ralph Wheland	Centre County Farm Bureau
Mr. Robert Donaldson	Centre County Office of Planning
Mr. Dan Pennick	Centre County Office of Planning
Mr. John Eich	Formerly with Centre County Office of Planning
Mr. Shawn McLaughlin	Formerly with Centre County Office of Planning
Ms. Beth Thoma	Centre County Office of Planning
Ms. Renee Miller	Centre Hall Borough
Mr. Robert Crum	Centre Regional Planning Agency
Mr. Robert Jacobs	Formerly with Centre Regional Planning Agency Currently with Centre County Office of Planning
Mr. John F. Coleman, Jr.	Chamber of Business & Industry
Mr. Gerald Rogers	Citizen
Ms. Darleen Weener	Clearwater Conservancy
Mr. Lee Shields	College Township
Mr. John R. Everhart, Jr.	Community, State College Young Farmers
Mr. Robert Crassweller	Ferguson Township
Mr. David Piper	Halfmoon Township
Ms. Sherene Hess	League of Women Voters of Centre County
Mr. Donald Ammerman	Milesburg Borough
Mr. Jacob Bliek	PA Department of Corrections
Mr. Dan Alters	PA Department of Environmental Protection
Mr. David B. Jostenski, P.E.	PA Department of Environmental Protection
Mr. Jim Bathurst	PA Department of Transportation District 2-0
Mr. David Bumann	PA Fish and Boat Commission
Mr. Lewis McGill	PA State Correctional Institution - Rockview
Ms. Karen Drosnes	Patton Township
Senator J. Doyle Corman, Jr.	Pennsylvania State Senate
Mr. Robert Cooper	Pennsylvania State University - OPP
Mr. B. K. Bastress	Potter Township
Mr. Gert Aron	Spring Creek Watershed Community
Ms. Barbara Fisher	Spring Creek Watershed Community
Mr. Mark Ralston	Spring Creek Watershed Community
Mr. Joseph Senft	Spring Creek Watershed Community
Mr. John Auman	Spring Township
Mr. Mark Whitfield	State College Borough
Mr. Max Gill	State College Borough Water Authority
Mr. Melvin Brown	Walker Township

TAC Member Affiliation

Mr. Doug Weikel	Benner Township Engineer
Mr. Richard Lahr	Benner Township Supervisor
Mr. James Coslo	Centre County Conservation District
Mr. Robert Donaldson	Centre County Office of Planning
Mr. Dan Pennick	Centre County Planning Office
Mr. John Eich	Formerly with Centre County Planning Office
Mr. Shawn McLaughlin	Formerly with Centre County Planning Office
Ms. Beth Thoma	Centre County Planning Office
Mr. Robert Crum	Centre Regional Planning Office
Mr. Robert Jacobs	Formerly with Centre Regional Planning Office Currently with Centre County Planning Office
Mr. C. Chris Exarchos	College Township Council
Mr. Kent Baker	College Township Engineer
Mr. John R. Everhart, Jr.	Community State College Young Farmers
Mr. David J. Modricker	Ferguson Township Director of Public Works
Mr. Walt Ebaugh	Hydrogeologist
Mr. Mark Ralston	Hydrogeologist
Mr. David B. Jostenski, P.E.	PA Department of Environmental Protection
Mr. Dan Alters	PA DEP Water Quality
Mr. David Spotts	PA Fish and Boat Commission
Mr. David Bumann	PA Fish and Boat Commission
Mr. James Bathurst	PA Department of Transpiration District 2-0
Mr. Doug Erickson	Patton Township Engineer
Dr. Gert Aron	PSU - Professor Emeritus Hydrology
Dr Robert Carline	PSU - Professor of Fish & Wildlife
Dr. C. Andrew Cole	PSU - Environmental Research
Dr. Brian Dempsey	PSU - Assoc. Professor Civil Engineering
Dr. William A. Dunson	PSU - Professor Emeritus of Biology
Dr. Barry Evans	PSU - Land Water Research
Mr. John Gaudlip	PSU - Office of Physical Plant
Ms. Caren Glotfelty	PSU - Goddard Professor Agric. Sciences
Mr. Neil Korostoff, ASLA	PSU - Dept. Landscape Architecture
Dr. David Long	PSU - Professor Emeritus Civil Engineering
Dr. Richard Parizek	PSU - Professor Geology
Dr. Rob Brooks	PSU - Professor Wildlife and Wetlands
Dr. J. Greg Ferry	PSU - Professor Biochemistry & Molecular Biology
Dr. Ann Fisher	PSU – Senior Scientist
Dr. Jon Hall	PSU – Associate Professor Soil Chemistry
Ms. Amy Story	State College Borough Engineer
Mr. Cory Miller	University Area Joint Authority

Chapter 2

WATERSHED CHARACTERISTICS AND DATA SUMMARIES

The Spring Creek Watershed is located in the southern part of Centre County and is contained within the following fourteen (14) municipalities:

BOROUGHS	TOWNSHIPS
Bellefonte Borough	Benner Township
Centre Hall Borough	Boggs Township
Milesburg Borough	College Township
State College Borough	Ferguson Township
	Halfmoon Township
	Harris Township
	Patton Township
	Potter Township
	Spring Township
	Walker Township

Existing and Future Land Uses

Comprehensive land use plans for the municipalities within the Spring Creek Watershed are collected at the Centre County Planning Office (CCPO) and the Centre Regional Planning Agency (CRPA). These plans, along with current zoning requirements, were used as the basis for determining existing land uses and predicting the future development conditions within the Watershed. The existing impervious coverage for the Watershed is approximately 12%. Current land use designations for the Watershed are as follows:

LAND USE	ACRES	PERCENTAGE
Agriculture	33,795	36
Commercial	1,117	1
Forest/Water	34,767	37
Industrial/Mines	1,609	2
Public/Institutional	1,350	2
Recreation	1,868	2
Residential	8,755	10
Transportation/Utilities	4,996	5
Vacant	4,846	5
TOTAL	**93,103**	**100**

As shown, existing land use in the Watershed is primarily forest and agriculture. The distribution of land uses throughout the Watershed is shown in Exhibit 2-1 (exhibits are located at the end of this Chapter). Large portions of the forested land are Bald Eagle State Forest, Rothrock State Forest, and State Game Lands No. 176. Agricultural land dominates the valleys outside of the developed areas of State College and Bellefonte. Major farming enterprises include dairy, grain, and produce.

The Spring Creek Watershed has been experiencing enormous growth during the past few decades. With 70% percent of the County's population residing in the Watershed and the infrastructure development and improvements planned in the area, growth will continue to concentrate within the Watershed and slowly expand outward. It is estimated that the impervious coverage of the Watershed could increase by as much as 20% over the next ten years. Exhibit 2-2 illustrates a projection of future growth boundaries within the Watershed. This map is based on land use projections made by CCPO and CRPA.

Municipal Ordinances

The existing Land Use and Land Development Ordinances in place in each of the fourteen (14) municipalities are listed in Table 2-1. As shown, Benner Township, Centre Hall Borough, Milesburg Borough, Potter Township, and Walker Township do not have local land development ordinances. Instead, these municipalities fall under the Centre County Subdivision and Land Development Ordinance.

An evaluation of each municipal ordinance was conducted as a part of the development of the Spring Creek Watershed Stormwater Management Plan. The purpose of this evaluation was to identify the municipalities' development objectives and current stormwater control procedures. In addition, the evaluation helped to determine the general extent to which each existing ordinance will have to be modified in order to accommodate implementation of the Spring Creek Watershed Stormwater Management Plan.

Table 2-1
Existing Municipal Ordinance Matrix
Spring Creek Watershed

Municipality	Stormwater Ordinance	Subdivision and Land Development Ordinance	Floodplain Management Regulation	Zoning Ordinance
Bellefonte Borough	Yes	Yes	Yes	Yes
Benner Township	Yes	No*	Yes	Yes
Boggs Township	No	Yes	No	Yes
Centre Hall Borough	No	No*	No	Yes
College Township	Yes	Yes	Yes	Yes
Ferguson Township	Yes	Yes	Yes	Yes
Halfmoon Township	Yes	Yes	Yes	Yes
Harris Township	Yes	Yes	Yes	Yes
Milesburg Borough	No	No*	No	Yes
Patton Township	Yes	Yes	Yes	Yes
Potter Township	Yes	No*	Yes	Yes
Spring Township	Yes	Yes	Yes	Yes
State College Borough	Yes	Yes	Yes	Yes
Walker Township	No	No*	No	Yes
Centre County	Yes	Yes	Yes	No

* Municipality falls under the Centre County Subdivision and Land Development Ordinance.

Soils Information

The Spring Creek Watershed consists predominately of soils that formed from residual and colluvial material weathered from limestone within the valley regions and residual and colluvium material weathered from sandstone and some intermixed shale on the ridges. The distribution of soil associations within the Watershed is illustrated in Exhibit 2-3. This distribution is based on information from the NRCS's State Soil Geographic Database (STASGO). STATSGO information is a generalization of detailed data from the State's County Soil Surveys. As a result, STATSGO data coordinates soil map units between the counties and creates one seamless database to form statewide coverage. This generalization explains why the soil association names and boundaries are slightly different in Exhibit 2-3 from what is found in the Centre County Soil Survey. At this time, soil association data from the Soil Survey is not available for digital mapping.

According to the Centre County Soil Survey (USDA, 1981), there are four soil associations within the Watershed that are formed from weathered limestone: Hagerstown-Opequon-Hublersburg, Morrison, Opequon-Hagerstown, and Murrill associations. The Hagerstown-Opequon-Hublersburg association comprises over 50% of the soils within the Watershed. These associations are characterized by level to sloping, well-drained soils underlain by limestone bedrock. The soil depth can vary greatly within these soil associations. These soil associations are found in the limestone valleys of the Watershed. According to TR-55, the soils are primarily classified as hydrologic soil group (HSG) B and C.

The ridge areas of the Watershed consist of two soil associations: Hazelton-Laidig-Andover and Ungers. Both associations are formed from weathered sandstone and are primarily classified as hydrologic soil groups B and C. The Hazelton-Laidig-Andover association is found in approximately 25% of the Watershed (predominantly mountain ridges). This association is characterized by well drained to poorly drained soils underlain by acid sandstone bedrock.

At this point, a cautionary statement needs to be made. Soil surveys and the data they contain are only a starting point from which to design a site plan. In Addition, soil survey mapping is limited by the scale at which the data is contained in the survey, which should be kept in mind for small sites. The soil survey is a planning level tool only, which contains excellent data that should be used to restrict development in some areas or help plan where more detailed study should be conducted. Unfortunately, the neglect of subsurface features such as fragipans or clay pans, which are well defined in the surveys, can cause flooding problems for buildings with or without upslope development. However, this type of information in the soil surveys is often neglected, while too much emphasis is placed on the use of Hydrologic Soil Groups (HSG).

The HSG was originally developed as an approximate average value used to estimate runoff potential. Four (4) HSG classifications are used to represent thousands of soils underlain by different geology in different regions of the United States. HSG were developed considering precipitation events that produced large flood events. The HSG were determined by “assuming that the soil surfaces were bare, maximum swelling had taken place, and rainfall rates exceeded surface intake rates” (USDA, 1993) and after prolonged wetting using the soil B horizons. This is almost never the case for most rainfall events, especially those events that are specifically

aimed to control BMPs. . In addition, the HSG has nothing to do with recharge and should not be used as an indicator of recharge or for any other purpose that it was not originally intended.

The all-to-frequent failure of residential septic leach beds in all types of soils (and HSG classifications) should lead one to be cautious of placing too much emphasis on Hydrologic Soil Groups or infiltration systems for large impervious developments to control radical changes in volume. When done, these systems need to be thoroughly investigated and engineered. The successful implementation of this Stormwater Management Plan relies heavily on the performance of on-site soils. Therefore, the agronomic descriptions in the Soil Survey should be verified prior to final design and/or construction in order to have the best benefit for all parties affected.

Soil investigations for the design of stormwater BMPs should include, but not be limited to, the following elements:

- Characterizing the extent (laterally and vertically) of the soil units onsite and their suitability for use for SWM, *during the overall site design* for the project. This will allow *assessment of soil resources prior to construction* and the understanding of soil properties for SWM design;
- Confirming NRCS soils mapping prior to commencing *hydrologic modeling*; e.g., is mapping correct or have soils been altered and/or compacted from previous activities;
- Characterizing soil morphology and physical properties which can allow an estimation of *soil hydraulic conductivity* or selection of suitable testing methods;
- Assessing the soil's ability to *attenuate stormwater contaminants;*
- Identifying soil redoximorphic features and presence of *shallow water tables;*
- Identifying/reserving the most suitable onsite soils to be used in the construction of *clay liners and wetland substrates, as well as created soil infiltrating mediums, etc;*
- Assisting with the siting and evaluating the suitability of *recharge and BMP structures* (basins, trenches, seepage beds, roof sumps, drainage swales, etc.) via the examination of soils morphology, texture, depths to limiting zones, etc.; and
- Completing an *assessment of karst landscapes* from a soils perspective by examining for evidence of settlement activity, manmade fills (which can disguise sinkholes or depressional features), etc.

Geology/Karst Areas/Significant Karst Features

According to the Geologic Map of Pennsylvania (PA DER, 1980), the Spring Creek Watershed is located in geologic formations that primarily date from the Ordovician and Cambrian eras. Limestone/Dolomite (karst) geologic formations are predominate within the valley areas of the Watershed, with the Gatesburg and Nittany formations being the most prevalent. Sandstone formations such as the Bald Eagle, Reedsville, and Juniata Formations primarily characterize the ridge areas. Exhibit 2-4 illustrates the carbonate and non-carbonate geologic regions within the Watershed. Identified fracture traces are also illustrated in Exhibit 2-4.

Approximately 80% of the Spring Creek Watershed is underlain by limestone and dolomite rock formations. These carbonate formations are characterized by karst topography, such as sinkhole development and cavernous areas. The karst features of the Watershed make the creek particularly susceptible to increased direct stormwater runoff and pollutant loadings. Four types of karst features and their approximate locations are illustrated in Exhibit 2-5. These features include caves, sinkholes, surface depressions, and surface mines. Exhibit 2-5 was compiled from available published records and is not intended to be a comprehensive compilation of all such features. However, it does indicate areas prone to the development of karst features.

Hydrologic Features

The Spring Creek Watershed encompasses an area of approximately 145.9 square miles and includes the following major tributaries: Big Hollow Run, Buffalo Run, Cedar Run, Galbraith Gap Run, Roaring Run, Slab Cabin Run, Logan Branch, and Thompson Run. Spring Creek flows into the Bald Eagle Creek at Milesburg. The Watershed is bounded by Bald Eagle Mountain to the northwest and Tussey Mountain to the south, and it includes a portion of Nittany Mountain to the east.

To serve as the basis for the hydrologic and hydraulic analysis of this study, the Watershed was broken down by major tributary drainage into nine (9) sub-basins. These sub-basins were further broken down into twelve (12) subwatersheds based on the location of monitoring stations along the creek and its tributaries. The subwatersheds are illustrated in Exhibit 2-6.

Flow Obstructions

Obstructions along channels limit flow capacity and can potentially cause significant ponding or diversion of water. Municipal officials identified Fifty-nine (59) significant hydraulic structures within the Spring Creek Watershed.

Table 2-2 lists the significant structures, their locations within the Watershed, their descriptions, and the identification number assigned to them by the corresponding municipalities. Although these structures are considered channel obstructions, the use of the term obstruction does not imply that the structure is deficient. Obstruction/Structure locations, identified by ID number, are illustrated in Exhibit 2-7.

The Capacities indicated in Table 2.2 were computed using inlet control assumptions with 1-foot of head above the top of the structure. These values should not be considered as limiting capacities. Where appropriate, a more complete analysis including hydrologic analysis and consideration of tailwater effects should be undertaken.

Table 2-2
Inventory of Significant Hydraulic Structures

Sub-Watershed	Location	Description	Estimated Capacity (cfs)	ID[1]
Spring Creek	Trout Road	Wooden Bridge	5,370	COL-1
Thompson Run	Rte 26	Box Culvert	1,500	COL-2
Spring Creek	Brush Valley Road		2,250	COL-3
Spring Creek	Talleyrand Park	RR Bridge	17,240	BEL-A
Spring Creek		Pipe Bridge	5,290	BEL-B
Spring Creek	Talleyrand Park	Suspension Bridge	12,670	BEL-C
Spring Creek	High Street	Bridge	12,170	BEL-D
Spring Creek	Lamb Street	Bridge	6,690	BEL-E
Logan Branch	Harrison Road (Gap Run)	12" Culvert Outflow Higher than Pipe	4	SPR-1
Logan Branch	Logan Branch, Hatchery to Cerro	Trees/Debris in Stream	NA	SPR-2
Logan Branch	Locust St./Jordan Ave.	Sinkhole full of debris from SR 144	NA	SPR-3
Slab Cabin Run	Branch Road	Span Bridge	2,160	SCB-1
Slab Cabin Run	Centre Hills Golf Course	Covered Bridge	980	SCB-2
Slab Cabin Run	Atherton Street	2 Span Bridge	5,650	SCB-3
Slab Cabin Run	Marilyn Ave	Abandon Bridge	580	SCB-4
Buffalo Run	Rte 550, 3300' West of Meeks	4'x5'	170	PAT-1
Buffalo Run	Deer Brook	6'6"	290	PAT-2
Buffalo Run	Adjacent to Meeks	48"	90	PAT-3
Buffalo Run	Adjacent to Meeks	48"	90	PAT-4
Buffalo Run	Adjacent to Meeks	36"	50	PAT-5
Buffalo Run	Adjacent to Meeks	36"	50	PAT-6
Buffalo Run	Under Meeks Lane	60"	160	PAT-7
Buffalo Run	US 322	7'4"x19'10"	1,510	PAT-8
Buffalo Run	Upstream of PAT-10	4'x6' Box	200	PAT-9
Buffalo Run	Stevenson Road	4'6"x12'	470	PAT-10
Buffalo Run	Rte 550	4'x12' Box	400	PAT-11
Buffalo Run	Between Rte 550 & US 322	3'x8' Box	180	PAT-12
Buffalo Run	US 322	5'x13' Box	590	PAT-13
Buffalo Run	Upstream of Stevenson Rd	5'4"x23' Bridge	1,120	PAT-14
Buffalo Run	Stevenson Road	4'4"x9' Box	330	PAT-15
Buffalo Run	Julian Pike	84" Culvert	340	PAT-16
Buffalo Run	Downstream of Julian Pike	3x 36" Culvert	140	PAT-17
Buffalo Run	Downstream of PAT-17	7'x4'8" Arch	180	PAT-18
Buffalo Run	Downstream of PAT-18	39" Culvert	60	PAT-19
Buffalo Run	900' Downstream of PAT-19	48" Culvert	90	PAT-20
Buffalo Run	1800' Downstream of PAT-20	3'x16' Bridge	370	PAT-21
Buffalo Run	Rte 550, 900' downstream of PAT-20	7'x20' Bridge	1,430	PAT-22
Buffalo Run	1250' Downstream of PAT-21	8'x5' Arch	240	PAT-23
Buffalo Run	Fillmore Road	6'x41' Bridge	2,370	PAT-24

Sub-Watershed	Location	Description	Estimated Capacity (cfs)	ID[1]
Spring Creek	Rock Road	2-38"x56" Culvert	160	BEN-1
Spring Creek	Rock Road	Bridge	10,250	BEN-2
Spring Creek	Rock Road	Weir Structure	810	BEN-3
Spring Creek	Rock Road Culvert	36"x60" CMP	70	BEN-4
Spring Creek	Buffalo Run Road	Bridge	20,180	BEN-5
Spring Creek	Fisherman's Paradise	8'x2' RCB	110	BEN-6
Spring Creek	Buffalo Run Rd & Fish Paradise	60" RCP	160	BEN-7
Spring Creek	Buffalo Run Rd & Slaugh	30"x36" RCP	40	BEN-8
Spring Creek	Water St.	Abandon Bridge A	3,420	BEN-9
Spring Creek	Spring Creek Rd & Paradise Rd	10'x5' Bridge	450	BEN-10
Spring Creek	Spring Creek Road	Bridge	3,790	BEN-11
Spring Creek	Fisherman's Paradise @ End of Spring Creek Road	Bridge	2,250	BEN-12
Spring Creek	Spring Creek Rd & Barns Lane	Conc. Bridge	7,060	BEN-13
Big Hollow	Rock Rd over Big Hollow	24" CMP	20	BEN-14
Spring Creek	Rock Rd below Airport Pond		50	BEN-15
Buffalo Run	Fillmore Road	Bridge	2,370	BEN-16
Buffalo Run	Continental Courts	2-84" CMP	680	BEN-17
Logan Branch	Rte-26	48" CMP	90	BEN-18

[1]COL – College Twp., BEL – Bellefonte Boro., BEN – Benner Twp.,
PAT – Patton Twp., SCB – State College Boro., SPR – Spring Twp.

Stormwater Related Problems and Possible Solutions

During the initial stages of preparing this Stormwater Management Plan, a questionnaire was distributed to each municipality in the Spring Creek Watershed to identify stormwater-related problems. The questionnaire asked each municipality to describe the problem areas and their probable causes as well as mark the area's location on a map. The municipalities identified a total of forty-nine (49) problem areas. These stormwater problem areas are illustrated in Exhibit 2-7.

Most of the reported stormwater problems result in roadway or localized flooding in various areas of the Watershed. Erosion and sedimentation are also frequent problems throughout the Watershed. Causes of these problems include mountain runoff, development near drainage-ways, and insufficient stormwater management systems. Table 2-3 lists the problem areas, possible causes, and typical frequencies of occurrence as reported by the respective municipalities.

In most of these cases, there is not an easy and quick solution to the stormwater problem. Most of the problems have existed for many years and would require significant alteration to the area's existing stormwater management system to correct them.

Existing and Proposed Flood Control Projects

There are no known major existing or proposed flood control projects within the Spring Creek Watershed.

Existing and Proposed Stormwater Control Facilities

The Watershed municipalities identified a total of forty-seven (47) significant stormwater control facilities throughout the Spring Creek Watershed. The locations of these facilities are shown in Exhibit 2-8. Most of these facilities are located in the developed areas of Patton, Ferguson, and College Townships. Several are also located near Bellefonte Borough. The largest stormwater management facility in the Watershed is referred to as the Bathgate Detention Pond and is located on the northeast side of the Penn State Campus. This wet pond facility drains most of the Penn State Campus and State College Borough.

In addition, areas within the Watershed served by storm sewer were identified. This information is shown in Exhibit 2-9.

Topographic Mapping

As part of a major GIS project, Centre County prepared digital five-foot contour mapping for most of the County's built-up areas. These contour maps are available for the majority of the Spring Creek Watershed and can be found at the Centre County GIS Office or Penn State's Land Analysis Lab (LAL).

Table 2-3
Inventory of Stormwater-Related Problem Areas

ID	Municipality	Watershed	Location	Type	Cause	Frequency
P-1	College Township	Spring Creek	Trout Road Wooden Bridge	Roadway Flooding		Less Than 1 time per year
P-2	College Township	Spring Creek	Well Field near Spring Creek Park	Stream Flooding		Less Than 1 time per year
P-3	College Township	Slab Cabin Run	Best Western - Branch Road	Parking Lot Flooding		Less Than 1 time per year
P-4	College Township	Slab Cabin Run	E. Branch Rd & Woodside Dr.	Roadway Flooding		1 time per year
P-5	College Township	Spring Creek	Oak Hall Farm	Flooding & Erosion		Less Than 1 time per year
P-6	College Township	Spring Creek	Lemont	Localized Flooding	Mountain Runoff	1 time per year
P-7	College Township	Spring Creek	Mt Nittany Terrace	Field & Mt. Runoff	Field & Mt Runoff	1 time per year
P-8	College Township	Big Hollow	Park Ave. & Fox Hollow Rd.	Roadway Flooding	Inadequate Drainage System	1 time per year
P-9	College Township	Big Hollow	Near Township Boundary	Roadway Flooding		Less Than 1 time per year
P-10	College Township	Spring Creek	Houserville Drainage-way	Private Property	Development in Drainage-way	1 time per year
PSU-3	College Township	Big Hollow	Downstream of Oakwood	In Channel Flooding	Upstream Development	
PSU-4	College Township	Big Hollow	Park Ave/ Beaver Stadium	Intersection Flooding	Insufficient Capacity	
PSU-5	College Township	Big Hollow	PFJA Interceptor Crossing	Numerous Sinkhole Develop	Sewer Crossing above ground	
PSU-6	College Township	Slab Cabin	Downstream Bathgate Basin	Overland Flooding & Erosion	Concentrated Pond Flows	
PSU-1	Ferguson Township	Big Hollow	North Corl & Underpass	Roadway Flooding	No positive drainage	
FER-A	Ferguson Township	Slab Cabin	Shadow Oaks	Stream & Roadway Flooding	Inadequate Drainage System	More than 1 time per year
FER-B	Ferguson Township	Big Hollow	N. Atherton	Roadway Flooding	Upstream Development	Less than 1 time per year
FER-C1	Ferguson Township	Big Hollow	Glen View to Corl Street	Roadway Flooding	Inadequate Drainage System	More than 1 time per year
FER-C2	Ferguson Township	Big Hollow	Glen View to Corl Street	Roadway Flooding	Upstream Development	More than 1 time per year

ID	Municipality	Watershed	Location	Type	Cause	Frequency
FER-C3	Ferguson Township	Big Hollow	Glen View to Corl Street	Roadway Flooding	Inadequate Drainage System	More than 1 time per year
FER-C4	Ferguson Township	Big Hollow	Glen View to Corl Street	Roadway Flooding	Upstream Development	More than 1 time per year
FER-D	Ferguson Township	Big Hollow	Park Hills Drainage-way	Roadway & Sediment-ation	Inadequate Drainage	More than 1 time per year
H-1	Harris Township	Spring Creek	Elks Club Entrance	Private Property	Insufficient Capacity	
H-2	Harris Township	Spring Creek	Sharer Road near Spring Creek		Logging - Runoff Problems	
H-3	Harris Township	Cedar Run	Smith Lane	Erosion Problems (East side)	Erosion	
H-4	Harris Township	Cedar Run		Runoff Problems	Mountain Runoff	
H-4	Harris Township	Cedar Run		Runoff Problems	Mountain Runoff	
H-5	Harris Township	Spring Creek	Rte 322 & 45		Debris in Channel	
H-6	Harris Township	Spring Creek	Harris St & Ishler Ave.	Stormwater Problems		
H-7	Harris Township	Spring Creek	W. Crestview Ave	Shoulder washout	Field & Mountain Runoff	
H-8	Harris Township	Spring Creek	Academy Street	Runoff Problems	Mountain Runoff	
H-9	Harris Township	Spring Creek	Church Street Extended	Stormwater Problems	Construction on Hillside	
H-10	Harris Township	Roaring Run	Mountain Road	Road Washout	Mountain Runoff	January 1996
H-11	Harris Township	Spring Creek	Kennard Road	Concern of potential flooding	Future Development	
SPR-B1	Spring Township	Logan Branch	Ishlers Field to 144	Stream/ Roadway/ Sediment-ation	Inadequate SWM system	More than 1 time per year
SPR-B2	Spring Township	Logan Branch	Ishlers Field to 144	Stream/ Roadway/ Sediment-ation	Inadequate SWM system	More than 1 time per year
SPR-C	Spring Township	Logan Branch	Phantom Lake	Roadway Flooding	Periodically Appears	More than 1 time per year

ID	Municipality	Watershed	Location	Type	Cause	Frequency
SPR-E	Spring Township	Logan/Spring	Blanchard St.	Roadway Flooding	Inadequate system/SWM	More than 1 time per year
SPR-F	Spring Township	Logan Branch	Black Hawk Road	Roadway Flooding	Springs along/through Road	More than 10 times per year
SPR-G1	Spring Township	Logan Branch	Weaver Hill Rd.	Sediment-ation/SW Pollution	Dry Drainage Sedimentation	More than 10 times per year
SPR-G2	Spring Township	Logan Branch	Blanchard St.	Sediment-ation/SW Pollution	Dry Drainage Sedimentation	More than 10 times per year
SPR-G3	Spring Township	Logan Branch	Irish Hollow Rd.	Sediment-ation/SW Pollution	Dry Drainage Sedimentation	More than 10 times per year
SPR-H	Spring Township	Logan Branch	Gap Run Sinkhole (Harr Rd.)	Stream/ Roadway/ Sediment-ation/ SW NPS	Sinkhole Overflow	Less than 1 time per year
SCB-A	State College	Big Hollow	Corl Street	Roadway/ Sinkhole	Flow Obstruction/ Inadequate	More than 1 time per year
SCB-B	State College	Slab Cabin Run	Highland Alley	Roadway Flooding	Inadequate SWM system	More than 1 time per year
SCB-C	State College	Slab Cabin Run	Holly Alley	Roadway Flooding	Inadequate SWM system	More than 1 time per year
SCB-D	State College	Slab Cabin Run	Saxton & Westerly Parkway	Roadway Flooding/ Sinkhole	Flow Obstruction/ Inadequate	Less than 1 time per year
PSU-2	State College Borough	Slab Cabin	Atherton @ Bus Station	Roadway Flooding		

Aerial Photographs

Digital ortho-photographs are available for the entire Spring Creek Watershed at the Centre County GIS Office or at Penn State's Land Analysis Lab (LAL. These maps were produced at a scale of one (1) inch to two hundred (200) feet and can be used as a data layer in a GIS system.

Streamflow and Precipitation Data

The U.S. Geological Survey maintains three continuous surface water-gaging stations within the Spring Creek Watershed: Spring Creek at Houserville (58.5 sq mi), Spring Creek near Axemann (87.2 sq mi), and Spring Creek at Milesburg (145.3 sq. mi). The Houserville gage is a water-stage recorder that has been in operation since November 1984. The period of record for the Axemann gage is from October 1940 to present. The Milesburg gage has been in operation since May 1967. In addition, the Spring Creek Watershed Community, through the Clearwater Conservancy, is collecting continuous stream flow and temperature data at nine additional sites throughout the Watershed. This data collection effort began in spring 1999, and the collected data are available from the Clearwater Conservancy.

Daily precipitation data were provided by the Penn State Meteorology Department from its gage located at the Walker Building on campus. The Meteorology Department also supplied hourly precipitation data from its rainfall gage in Rock Springs, adjacent to the Watershed. The data collected from these gages and sources, along with local precipitation records, were used to calibrate the hydrologic models used in this study.

Stream and precipitation gage locations are illustrated in Exhibit 2-6.

In addition, the Pennsylvania Department of Transportation (PennDOT) Field Manual of Storm Intensity-Duration-Frequency (IDF) Charts (1986) were used to estimate the depth of precipitation expected for certain design storms. The Spring Creek Watershed is in PennDOT Region 2, so data from the IDF Charts for Region II were also used in the modeling efforts. Table 2-4 below shows the 24-hour design storm depths for the 1- through 100-year return period storms. On average, the Watershed receives approximately 40 inches of precipitation per year.

Table 2-4
24-Hour Design Storm Depths

Return Period (years)	24-Duration (inches)
1	2.2
2	2.6
5	3.0
10	3.5
25	4.2
50	4.7
100	5.3

Floodplain Information

Flood Insurance Rate Maps for the streams within the Spring Creek Watershed are available from the Federal Emergency Management Agency (FEMA). Other sources include municipal offices and Centre County Planning Office. Digital mapping based on the FEMA maps is available from Pennsylvania Spatial Data Access (PASDA). Either FEMA or PASDA maps can be used to identify floodplains within the Watershed. Approximate floodplain and wetland boundaries are illustrated in Exhibit 2-10.

Water Quality Data

General water quality data used in preparing this Plan was collected from EPA, PA DEP, the Center for Watershed Protection (CWP), and the Clearwater Conservancy. Most of the EPA information consisted of data from the National Urban Runoff Program (NURP). DEP and CWP provided much of the information regarding stormwater BMPs.

The Spring Creek Watershed Community through the Clearwater Conservancy is collecting base flow water quality data. Chemical sampling has been on-going since spring 1999 at the twelve (12) gage sites established throughout the Watershed (see Exhibit 2-6). Flow data is also being collected but was not available for inclusion in this Plan.

Engineering and Planning Studies

Numerous engineering and planning studies related to stormwater have been performed within the Spring Creek Watershed in recent years. Many of these studies have been performed in conjunction with The Pennsylvania State University, the Clearwater Conservancy, and local consultants. The most expansive listing of these and other local studies can be found through the Clearwater Conservancy. The Conservancy maintains a large database of resources pertaining to many aspects of the Watershed. Refer to the Reference list in Appendix D for a listing of the studies that were used in the development of this Plan.

In addition to watershed-specific studies, several general stormwater runoff, water quality, and karst topography studies performed by PA DEP, EPA, other governmental agencies, and various professionals were consulted during the preparation of this Plan. For example, a portion of the information used in the Water Quality section of the Plan came from data collected during EPA's National Urban Runoff Project and various studies performed by the Chesapeake Bay Program. Studies performed on other Pennsylvania watersheds by PA DEP and the Susquehanna River Basin Commission also contain useful information. For karst topography issues, several research papers from the Hydrogeology and Engineering Geology of Sinkholes and Karst conference proceedings were consulted. Refer to the Bibliography list in Appendix D for a more detailed listing of studies related to stormwater and karst topography.

Chapter 3

TECHNICAL STANDARDS FOR STORMWATER MANAGEMENT

This chapter includes a discussion of the technical standards and criteria, that apply to the development of a Stormwater Management Plan (SWMP) for all land development activities within the Watershed. Background and technical analysis supporting these criteria can be found in Appendix E.

All regulated activities in the Spring Creek Watershed that do not qualify under one of the exemptions identified in the following section shall submit a SWMP to the governing municipality for review. This plan must be consistent with the technical standards of this Chapter. These criteria shall apply to the total proposed development, even if development is to take place in phases.

Exemptions

Activities identified below are exempt from the requirement to submit a Stormwater Management Plan to the governing municipality for review. However, items A, C, and D will require documentation to show that the site qualifies for the exemption. Exemption shall not relieve the applicant from implementing such measures as are necessary to protect health, safety, and property. These measures include adequate and safe conveyance of stormwater on the site and as it leaves the site. This exemption shall not relieve the applicant from meeting the requirements for water quality and ground water recharge, special requirements for high quality (HQ) and exceptional value (EV) watersheds (DEP Chapter 93 and anti-degradation requirements), and water quality and recharge volume requirements presented later in this chapter.

A. All development activities having impervious coverage of less than 10% of the total site area, up to a maximum impervious area of 20,000 square feet. However, adequate and safe conveyance of stormwater from the site must be provided. For developments that are to be constructed in phases, the sum of all final phases must be considered in

establishing exemption eligibility. Impervious cover shall include, but not be limited to, any roof, parking or driveway area, new streets, sidewalks, and bikeways.

B. Land disturbance associated with the construction or alteration of one- and two-family dwellings, provided that the disturbance does not alter any stormwater condition beyond the boundaries of the lot or alter provisions of a previously approved Stormwater Management Plan for the lot or encompassing subdivision. Multiple (>2) lot subdivisions cannot be exempted.

C. Any site less than one (1) acre in size that decreases the total site impervious area following development, and:

- Is not located within a recognized sensitive area (as defined in the following section);
- is not defined as a water quality sensitive (WQS) development (as defined in the following section); or
- is not located in an area where existing downstream stormwater problems are known to occur (the Municipal Engineer shall make the final determination as to pre-existing problems, but the municipality must have supporting documentation of past problems).

D. In addition, the Municipal Engineer may waive the requirement to prepare a stormwater management plan for sites larger than 1.0 acre for which the overall site impervious area is being decreased, and which meets the other conditions identified above.

The diversion or piping of any natural or man-made stream channel and/or the installation of stormwater management facilities or modifications thereto cannot be exempted. These activities always require the submission of a Stormwater Management Plan. Exemptions A and B cannot be combined for use with small residential subdivisions.

In addition to the general exemptions identified above, exemptions related to specific technical criteria are identified where applicable in the following sections.

Sensitive Areas and Developments

Sensitive areas and water quality sensitive developments require special consideration with regard to stormwater management.

Sensitive areas are defined as those areas that, if developed, have the potential to cause catastrophic loss to a Water Authority well field. These areas consist of the delineated 1-year zone of contribution and direct upslope areas tributary to the wells. Exhibit 3.1, located at the end of this chapter, illustrates defined sensitive areas within the Watershed. Municipalities may update the sensitive area boundaries based on new research or studies as required.

Water Quality Sensitive (WQS) developments are defined as land development projects that have a high potential to cause catastrophic loss to local water quality and could potentially threaten ground water reservoirs. The following is a provisional list of WQS developments. This list may be amended at the discretion of the local municipality.

- Vehicle fueling stations
- Industrial manufacturing sites*
- Salvage yards
- Recycling centers
- Hazardous material storage areas*
- Interstate highways

* The Municipal Engineer will make the determination relative to what constitutes these classifications on a case-by-case basis. The Pennsylvania DEP wellhead protection contaminant source list shall be used as a guide in these determinations. Industrial manufacturing sites and hazardous material storage areas must provide NPDES SIC codes.

Performance Standards

A. General - Post-development rates of runoff from any regulated activity shall not exceed the peak release rates of runoff prior to development for the design storms specified.

B. Sensitive Area District Boundaries – The location of Sensitive Areas or Sensitive Area Districts (SAD) within the Watershed are illustrated on an official map, that is available for inspection at any municipal office. A reduced scale copy of this map is included as Exhibit 3.1 at the end of this chapter. The exact location of the boundaries of sensitive areas are set by drainage areas tributary to each of the points of interest as illustrated in Exhibit 3.1 and Exhibit 3.3. The exact location of these boundaries, as they apply to a given development site, shall be determined using mapping at a scale that accurately defines the limits of the sensitive area. If the project site is within the sensitive area (in whole or in part), 2-foot contour interval mapping shall be provided to define the limits of the Sensitive Area. If the project site is adjacent to, but within 500 linear feet of a defined Sensitive Area, a 5-foot contour interval map defining the limits of the Sensitive Area shall be included in the Stormwater Management Plan to document the site's location relative to the sensitive area.

C. Sites Located in More Than One (1) District - For a proposed development site that is traversed by a SAD boundary, the design criteria for sensitive areas must be applied if post-development runoff is directed towards the sensitive area.

D. Off-Site Areas - Off-site areas that drain from sensitive areas through a proposed development site that is located entirely in a non-sensitive area are not required to use or apply the sensitive area criteria.

E. Site Areas - Where the site area to be impacted by a proposed development activity differs significantly from the total site area, only the proposed impact area shall be subject to the design criteria.

F. "Downstream Hydraulic Capacity Analysis" - Any downstream or off-site hydraulic capacity analysis conducted in accordance with these standards shall use the following criteria for determining adequacy for accepting increased peak flow rates:

1. Natural or man-made channels or swales must be able to convey the post-development runoff associated with a 2-year return period event within their banks at velocities consistent with protection of the channels from erosion. Acceptable velocities shall be based upon criteria included in the DEP *Erosion and Sediment Pollution Control Program Manual*.

2. Natural or man-made channels or swales must be able to convey the post-development 25-year return period runoff without creating any hazard to persons or property.

3. Culverts, bridges, storm sewers or any other facilities that must pass or convey flows from the tributary area must be designed in accordance with DEP Chapter 105 regulations (if applicable) and, at a minimum, pass the post-development 25-year return period runoff.

4. It must be demonstrated that the downstream conveyance channel, other stormwater facilities, roadways, or overland areas must be capable of safely conveying the 100-year design storm without causing damage to buildings or other infrastructure.

5. Where the downstream conveyance channel or other facility is located within a special flood hazard area (as documented on the municipal Flood Insurance Rate Map), it must be demonstrated that the limits of said flood hazard area are not increased by the proposed activity.

6. Stormwater management ponds that fall under the DEP Chapter 105 Criteria of a “Dam” must meet the criteria within Chapter 105.

G. Sustainable development and Best Management Practice techniques (as outlined in Chapter 4) should be applied for stormwater management wherever possible.

Design Criteria and Calculation Methodologies

Design criteria and calculation methodologies have been classified by functional group for presentation as follows: 1) peak runoff rate discharge requirements; 2) stormwater pond capture volumes; 3) recharge volumes; 4) storm drain design including conveyance, channel protection, and stability; and 5) water quality standards.

These criteria and calculation methodologies have been developed to simplify stormwater management designs, unify methods, remove model parameter subjectivity, remove improperly used methods, and to ensure that stormwater management decisions are based more realistically on hydrologic processes. In addition, common sense should always be used as a controlling criteria.

These standards provide consistent and process-oriented design procedures for application by land development professionals. It is recognized that in an attempt to generalize the computational procedures, assumptions have been made which on some occasions may be violated. If such a violation is identified, alternate standards and procedures may be applied. Both the violation and the alternate procedures to be applied must be documented by a qualified hydrologist or hydrogeologist. Any request for use of alternate standards or procedures under this provision must be agreed to by the local Municipal Engineer prior to formal submission of plans for consideration by the municipality.

A flow chart documenting the stormwater management design process is provided in Exhibit 3.2 at the end of this chapter. The Application of these design criteria and methodologies is documented through examples in Appendix C.

A. Peak Runoff Rate Control

1.a Any site where the increase in post-development peak runoff rates is determined to be negligible by the Municipal Engineer is exempt from the requirement to

provide stormwater detention. In support of this exemption, it must be shown that the downstream conveyance systems have adequate capacity to convey the additional discharge without adversely affecting downstream properties. This does not exempt the requirement for implementation of designs for water quality, stormwater conveyance, and/or recharge as required. A Stormwater Management Plan and report documenting these design elements is also required. The Municipal Engineer shall use a 5% increase in peak discharge rate as a general benchmark for defining "negligible". However, depending on site specific conditions, a smaller or larger peak discharge rate may be appropriate. The final definition of "negligible" shall be at the Municipal Engineer's discretion.

Prior to using this exemption (and prior to any land development plan submission), the Design Engineer must provide written documentation and computations supporting the request for a waiver from peak runoff control requirements. The Municipal Engineer has the right to reject any plan that uses this assumption without prior approval of the Municipal Engineer. The intent of this exemption is to eliminate the need for multiple or "piggyback" detention facilities as a result of minor changes in imperviousness or land use upstream of existing stormwater control facilities.

1.b Sites less than five (5) acres in size that are located directly adjacent to the main stem of creeks or within the floodplain are not required to provide stormwater detention unless directed to do so by the Municipal Engineer as a result of a documented drainage problem. All other stormwater management standards must be implemented, including water quality, adequate stormwater conveyance, and/or recharge as required. The Municipal Engineer has the right to reject any plan that uses this exemption without prior approval of the Municipal Engineer.

2. Stormwater management analysis must be performed using the following models. The size criteria are based on drainage area size, which includes the site area and all off-site area draining across the development.

Up to 100 acres in size	NRCS's TR-55 or TR-20
Over 100 acres in size	NRCS's TR-20 or HEC-1 (HEC-HMS)

The Rational Method or Modified Rational Method may be used for any site less than or equal to two (2) acres in size without prior authorization from the Municipal Engineer. The Rational Method or Modified Rational Method may also be used for sites between two (2) and twenty (20) acres in size where the Municipal Engineer has approved the methods use. In this case the Design Engineer must make a written request to the Municipal Engineer explaining why the use of the Rational Method is more appropriate than the NRCS's methods for the site in question. The Design Engineer should keep in mind that the Rational Formula Methodology was not calibrated to account for the karst nature of the Spring Creek Drainage Basin; and therefore, its use should be limited to the special cases identified above. In addition, since the minimum discharge criteria are based on a calibration of the NRCS runoff model, their use is not appropriate if the Rational Method is used for runoff computations.

The Municipal Engineer has the right to reject any SWM design that uses hydrograph combinations with the Rational Method where the designer has not validated that the effects of the timing differences are negligible. In addition, the Municipal Engineer has the right to reject any SWM design that improperly uses the method for determining runoff volumes or does not properly apply the method.

More intensive, physically-based models may be used at the discretion of the Municipal Engineer but only for sites greater than 100 acres in size.

Commercial software packages that use the basic computational methods of TR-55 or TR-20 are permitted.

The NRCS models and methods recommended above are based on data collected from actual watersheds. In contrast to this, stormwater management analysis for land development activities is often conducted using property lines to

define drainage boundaries. Drainage areas based on property boundaries are not true watersheds and are referred to here as "hypothetical" drainage areas. It is known that these hypothetical drainage areas do not respond like natural watersheds. Peak runoff rates from hypothetical drainage areas are much smaller than comparable runoff rates from natural watersheds of the same size. Therefore, wherever possible, pre- and post-development stormwater analyses should be conducted for watersheds that are as nearly natural as possible.

It is noted that natural watershed boundaries should not be used where the relative size of the watershed compared to the site size would inappropriately distort the pre- to post-development runoff comparison. In these cases a hypothetical drainage area defined by the property boundary should be used as it allows for a better estimate of runoff changes directly downstream of the site. In addition, the designer should recognize that within the Spring Creek Watershed typical hypothetical drainage areas, in their pre-development or natural condition, do not produce surface runoff during minor to moderate rainfall events. Available hydrologic models do not accurately reflect this condition. This often results in post-development nuisance flooding since the models over-estimate the pre-development runoff magnitude.

3. Major natural drainage divides may not be altered without the prior consent of the Municipal Engineer.

4. Pre- and post-development stormwater management analysis shall be conducted using the following design storms:

1 year	2-year
10-year	100-year

For sites less than one (1) acre in total area that connect directly to existing storm sewer systems, surface or subsurface (underground) stormwater detention facilities only need to be designed to control storm events up to the design return period of the existing pipes (usually 10 years). However, it must be

demonstrated that adequate conveyance capacity (overland or within the existing storm sewer system) exists to ensure that flooding or damage from proposed releases will not exceed the existing potential for the system. If warranted by historic flooding in the tributary storm sewer system, the municipality may require more stringent criteria.

The Municipal Engineer may waive the requirement to detain the 100-year storm as long as the discharge is to a well defined, functioning conveyance system that does not currently exhibit flooding or other conveyance problems. The downstream conveyance system must be analyzed for the 100-year event to ensure that the proposed development will not increase flooding or damage to existing buildings and/or infrastructure.

5. The following 24-hour precipitation depths shall be used for stormwater management analysis for the entire Spring Creek Basin. These values override the use of TP-40 (the basis of the NRCS 24-hr precipitation maps).

Return Period	Precip. Depth
1-year	2.2 inches
2-year	2.6 inches
10-year	3.5 inches
25-year	4.2 inches
50-year	4.7 inches
100-year	5.3 inches

6. The NRCS's Type II precipitation distribution is required for all stormwater management analyses.

7. The NRCS's dimensionless unit hydrograph "k" factor shall be 484 for both pre- and post-development stormwater analyses.

8. All undeveloped areas are to be modeled as meadow or woods in good hydrologic condition. Existing impervious areas may be modeled as being impervious for pre-development conditions. The only exception is areas that are

actively in agricultural production (non-fallow). The designer may model the watershed using the actual agricultural land use/cover condition to show that increases from the site as compared to the all meadow condition are negligible.

9. The NRCS's curve number (CN) shall be used as the rainfall to runoff transformation parameter for all stormwater management analyses.

10. Curve numbers should be rounded to tenths for use in pre-packaged hydrologic models. It should be recognized that the CN is only a design tool with a large degree of statistical variability. For large sites, CN's should realistically be rounded to the nearest whole number.

11. The NRCS's method to determine unconnected impervious area adjustments for CN can be used for distinctly defined impervious land areas that flow onto pervious areas in a dispersed manner. The method may only be used to calculate runoff from site impervious areas that actually flow across pervious areas. The method cannot be applied to the entire site using average weighted CN values.

12. Soils underlain by carbonate geology (limestone or dolomite) shall be classified as a hydrologic soil group (HSG) B soil for both pre- and post-development conditions regardless of the NRCS or Soil Survey's description, except for the following two conditions:

    a) Compacted structural fill areas shall use values/parameters associated with HSG C soils, at a minimum, for post-development conditions regardless of the NRCS or Soil Survey's description. For most developments, compacted structural fill areas are under impervious surfaces, but some may include islands within parking areas, fringe land, etc. HSG C values/parameters shall also be applied to large projects that clear and compact building pad areas for later phases of development under an initial phase. The Municipal Engineer shall make the final determination as to what areas of a land development site constitute

compacted structural fill. The intent is to account for large compacted areas and not minor grading within lawn areas.

b) Soils identified as “on flood plains” or “on terraces above flood plains” in the Centre County Soil Survey will use the HSG as designated in the Soil Survey. Refer to Exhibit 3.4 for a list of the soils.

13. Soils not underlain by carbonate geology shall use the HSG as specified by the NRCS or Soil Survey’s description, except for the following two conditions:

a) Wooded areas on HSG C and D soils shall be treated as HSG B for pre-development conditions. Disturbed post-development wooded areas shall carry the NRCS or Soil Survey's defined HSG with a minimum HSG of B. The user is cautioned that this criteria is for peak runoff rate control (pre- to post-) for stormwater management purposes only. Professionals attempting to quantify runoff rates for large structures, such as bridges or dams, should use the NRCS published HSG.

b) Highly compacted structural fill areas shall use parameters associated with HSG C soils, at a minimum, for post-development conditions regardless of the NRCS or Soil Survey’s description. For most developments these areas are normally covered with impervious surfaces, but some may include islands within parking areas, fringe land, etc. The use of parameters associated with HSG C soils also applies to areas that are compacted as structural fill for future phases of development. The Municipal Engineer shall make the final determination as to what areas of a land development site constitute compacted structural fill. The intent is to account for large compacted areas and not minor grading within lawn areas or small areas around buildings.

14. Areas draining to closed depressions must be modeled by removing the storage volume from the pre-development condition. The designer may assume that infiltration in the closed depression does not occur during a design runoff event.

Areas draining to closed depressions may also be used to adjust peak runoff rates to stormwater management ponds for the post-development analysis. This allowance has been developed to entice designers to intentionally design or leave in place small closed depressions that can reduce the total volume required from a stormwater management pond. The site designer is responsible to document downstream impacts if the closed depression were removed.

15. Drainage areas tributary to sinkholes shall be excluded from the modeled point-of-interest drainage areas defining pre-development peak flows. Assumptions that sinkholes spill-over during some storm events must be supported by acceptable documentation (as determined by the Municipal Engineer). In addition, the design professional must be aware that bypassing or sealing sinkholes will frequently result in downstream flooding and should not be done if existing downstream flooding already occurs. The site designer is responsible to document downstream impacts if the sinkhole were to stop taking stormwater runoff.

16. Ponds or other permanent pools of water are to be modeled by the methods established in the NRCS's TR-55 manual (1986). However, more rigorous documented methods are acceptable (as determined by the Municipal Engineer).

17. The NRCS antecedent runoff condition II (ARC II, previously AMC II) must be used for all simulations. The use of continuous simulation models that vary the ARC are not permitted for stormwater management purposes. In addition, prior to any continuous simulation model being used in the Spring Creek Basin for any other purposes, the model unit hydrograph must be modified for common events in addition to extreme events, based on an in-depth analysis of historical data from the basin.

18. The following Time of Concentration (Tc) computational methodologies shall be used unless another method is pre-approved by the Municipal Engineer:

- Pre-development – NRCS's Lag Equation.
- Post-development; commercial, industrial, or other areas with having impervious areas greater than 20% – NRCS's Segmental Method.*
- Post-development; residential, cluster, or other low impact designs less than or equal to 20% impervious area – NRCS's Lag Equation.*

The Tc is to represent the average condition that best reflects the hydrologic response of the area. For example, large impervious areas bordered by small pervious areas may not consider the effect of the pervious areas in the Tc computation. If the designer wants to consider the affect of the pervious area, runoff from the pervious and impervious areas must be computed separately, combining the hydrographs to determine the total runoff from the area.

Under no circumstance will the post-development Tc be greater than the pre-development Tc for any watershed or sub-watershed modeling purposes. This includes when the designer has specifically used swales to reduce flow velocities. In the event that the designer believes that the post-development Tc is greater, it will still be set by default equal to the pre-development Tc for modeling purposes.

* Refer to item number 28 regarding impervious area flashing (IAF).

19. The following post-development minimum discharges are permitted for use with the NRCS (CN) runoff model:

1-year return period	$Qp_{min} = 0.018\ (DA) + 0.2$
2-year return period	$Qp_{min} = 0.03\ (DA) + 0.4$
10-year return period	$Qp_{min} = 0.09\ (DA) + 1.0$

where: DA = the drainage area in acres
Qp_{min} = minimum allowable peak runoff rate in cfs

For return periods greater than 10 years, the minimum discharge shall be equal to the computed pre-development peak runoff rate.

The minimum discharge criteria above are not appropriate for use with the Rational Method. This is because these values were developed based on NRCS model corrections and do not actually represent a true physical process or discharge. However, common sense should be used by both the designer and reviewer in the evaluation of acceptable minimum discharges for use with the Rational Method.

The intent of the minimum discharge is to allow reasonable runoff release from a site when a hydrologic model has produced a pre-development runoff rate close to zero. The method is **NOT** permitted for areas that previously drained completely to sinkholes in order to bypass the sinkhole after development.

These minimum discharge values include the total of all stormwater management facilities' discharges and undetained area discharges. Undetained fringe areas (where the designer has made a realistic effort to control all new impervious areas) will have the peak runoff rates computed using the pre-development time of concentration for the drainage areas that are tributary to them. Undetained areas are those portions of the site that cannot be routed to a stormwater management facility due to topography, and typically include lower portions of pond berms or small areas around entrance drives. The site drainage areas used shall represent the pre-development condition, even if drainage areas are altered following development.

20. All lined stormwater management ponds in carbonate and non-carbonate areas must be considered impervious and may not be used as pervious areas for stormwater management computations. "Lined" here means lined with synthetic liners or Bentonite. All other compacted soil liners will be considered to be HSG D for hydrologic computations.

21. Stormwater management ponds that have a capture depth for the purposes of volume capture, water quality, or recharge shall assume a negligible discharge from these structures during design event routing. Only discharges from the primary principal spillway or emergency spillway need to be considered.

Discharges from subsurface drains that tie into a principal spillway should not be considered during design event routing.

22. Stormwater management ponds that have a capture depth for the purpose of capture volume, water quality, or recharge shall assume that the pond water quality or capture volume is full at the beginning of design event routing.

23. Stormwater management ponds must provide safe passage of the 100-year return period peak runoff rate assuming that all of the principal spillway orifices are fully clogged, and the principal spillway overflow is 50% clogged. A minimum of a 6-inch freeboard must also be maintained above the resulting "maximum" water surface elevations (W.S.E.). Any embankment emergency spillway can be assumed to be unclogged. SWM ponds with embankments completely made up of natural undisturbed soils (fully in “cut”) or where roadways act as the emergency spillway, are permitted. However, the Design Engineer must verify downstream stability and control.

24. All pre- and post-development comparisons of peak flows shall be rounded to tenths of a cfs. The intent here is to recognize the accuracy and precision limitations of hydrologic modeling procedures. Again, small differences between pre- and post-development discharge rates should be permitted when no negative downstream impacts will result.

25. The full Modified Puls routing method must be used for stormwater management pond analyses. Simplified methods of determining pond size requirements such as those in TR-55 (1986) can only be used for preliminary pond size estimates.

26. Pre-packaged hydraulic programs are not approved for the analysis of underground stormwater management facilities unless it can be verified that the program rounding subroutines used for the stage/storage data do not affect the results. This is because, for very small storage volumes, the program may round off the volume to a significant percentage.

27. Full supporting documentation must be provided for all stormwater management designs.

28. Designs must be checked for Impervious Area Flash (IAF). This check is used to determine if flooding may occur due to poor modeling choices specifically related to the time of concentration. This analysis requires that the watershed impervious area be modeled without the pervious areas. The time of concentration should also be determined from the impervious areas only. If the IAF analysis results in a higher peak runoff rate at a culvert or discharge from a pond, this higher rate must be used for the final design/comparison. The check will frequently yield higher values if a watershed's impervious area is located primarily near the watershed outlet or point of interest.

B. Pond Capture Volumes (Cv)

In order to minimize nuisance flooding from small precipitation events, a runoff capture volume will be required for all stormwater management ponds that do not discharge directly to natural, well-defined (with bed and banks) perennial streams. In general, natural well-defined streams in the Spring Creek Basin are limited to those delineated as USGS perennial streams. This should be treated as a guideline and not a steadfast rule. The final determination is at the discretion of the Municipal Engineer. The pond capture volume is a volume of runoff that will be retained in a pond below the elevation of any free surface principal spillway orifice. No principal spillway orifice (except those connected to subsurface drains), regardless of how small, shall be below the pond elevation equivalent to this volume.

The Centre County Conservation District (CCCD) receives numerous complaints regarding ponds that are located at the downslope edge of a property that result in discharging runoff onto downstream properties in an uncontrolled manner, or where no existing defined outlet channel exists. This is a very common problem in areas underlain by carbonate rock. These discharges can cause erosion and flooding downstream. While the Pond Capture volume is intended to minimize some of these negative effects, it cannot deter or reduce the impacts from poor design practices. Therefore, whenever

possible, the CCCD recommends that the designer consider the downstream morphological changes that may occur and, when possible, consider constructing conveyance systems to a stable natural channel. In some cases this may require cooperation between land owners. Likewise, designers and municipal officials should discourage development within these mild draws, especially where upslope development may occur.

The capture volume is defined as a runoff depth of 0.25 inches from all impervious areas tributary to the stormwater management facility. This volume will be allowed to infiltrate, evaporate, or dewater from a subsurface drain system connected directly to the facility's principal spillway. Supporting computations that show that 90% of the capture volume can dewater in a maximum of 72 hours must be provided. For surface ponds, the maximum depth of ponding for the capture volume shall be three (3) feet (a health and safety precaution). However, in areas under karst influence, a limiting maximum ponding depth of eighteen (18) inches is recommended. Designers may always increase the capture volume to a value greater than the identified standard as long as the ponding depth criteria are met.

To simplify computational requirements for design event analysis, designers do not need to calculate discharges from subsurface drains related to the capture volumes if the filter media is sand or material smaller than AASHTO 57 stone. The capture volume is to control runoff rates from impervious areas and is not intended specifically for water quality or recharge. However, pond designs that include a water quality or recharge volume that is greater than the required capture volume are assumed to have also met the required capture volume as long as the pond dewaters as required. In addition, the capture volume also acts as a credit for both the recharge and water quality volumes (refer to these respective sections).

Designs that rely on the natural infiltration of insitu soils must provide documentation supporting the infiltration rates used for analysis. Infiltration rates reported in the Soil Survey of Centre County or other published rates may be used at the discretion of the Municipal Engineer.

The pond capture volume should always be used when up-slope areas are developed and the pond's design creates a point discharge that did not previously exist.

Stormwater Management detention facilities that connect directly to storm drain pipe networks that discharge to natural well-defined channels do not require a capture volume.

C. Recharge Volumes (Rv)

The purpose of the recharge portion of the ordinance is two-fold. First, the recharge requirement is to mitigate the loss of groundwater recharge associated with the creation of impervious surfaces. In addition the recharge criteria is to mitigate the increase in runoff volume associated with the creation of impervious surfaces. This increase in runoff volume has significant impacts on downstream landowners. These impacts are most often exhibited in the form of increased nuisance flooding and channel or drainage-way erosion and instability. According to local Municipal Engineers and representatives of the Centre County Conservation District, these problems are of significant local concern. The magnitude of these problems increases with the percentage of impervious coverage created on a site.

Recharge mitigation shall be provided for runoff from all proposed impervious areas. The required recharge volume shall be computed as 0.5” of runoff from all proposed impervious areas. It is noted that lined detention ponds and compacted fill areas are considered to be impervious when calculating site impervious area for recharge considerations. In addition, land areas covered by paver blocks, pervious pavement, and other structural surface treatments which permit surface infiltration can be treated as pervious areas when calculating the site impervious area for recharge considerations as long as the structural infiltration practice is supported by sound design and appropriate construction specifications. The Municipal Engineer may require submission of supporting documentation prior to approving structural infiltration areas as pervious areas.

Refer to Appendix C for examples on the recharge credit and for differences between recharge and water quality.

The following can be used as credits against this required recharge volume:

1. Residential Roof Areas (detached, duplex, and townhome dwellings) and commercial /industrial buildings with roof areas less than 5,000 square feet can be removed from the computed impervious area when these roof areas are sumped to dry wells designed in accordance with the following minimum standard:

   SUMP DESIGN CRITERIA: To meet the recharge criteria, sump storage or voids volume shall be equal to 0.04 cubic feet per square foot of roof area (0.5 inch rainfall depth). If sump stone has a voids ratio of 40%, the total sump volume will be 0.10 cubic feet per square foot of roof area. When designed only to meet this recharge criteria, the maximum size for a single sump is 100 cubic feet, and the minimum sump surface area (A) to depth (D) ratio (A/D) must be a minimum of 4/1. The sump depth less any freeboard should not exceed 24". This roof sump standard shall apply unless the municipality has a separate roof sump standard for water quantity or peak control.

2. All or portions of driveways, roadways, and parking areas can be removed from the impervious area calculation when sheet flow from these areas is directed to undisturbed natural buffer/ filter areas or constructed filter strips. This flow must be dispersed as sheet flow as it crosses the buffer / filter area. Sheet flow velocities should be non-erosive as they cross the impervious area / filter interface.

   To ensure proper infiltration characteristics the natural soil profile within natural buffer / filter areas can not be disturbed during construction. Completely undisturbed natural recharge areas serve this function best. However, minor surface scaring, seeding, and landscaping is permitted in these areas as long as natural grades are not altered. In special cases, when approved by the Municipal Engineer, minor grading, combined with soil profile reconstruction may be

permitted in natural buffer / filter areas. In addition, the following standards apply to natural filter / buffer areas:

a. **Natural** filter / buffer areas must have a minimum width of five (5) feet or one-half of the impervious area drainage length immediately tributary to the buffer area, whichever is greater. This width is measured parallel to the direction of sheet flow.

b. To qualify for a recharge volume credit, the surface slope of natural filter / buffer areas must be conducive to recharge, and not result in flow concentration or erosion. To meet this intent, the surface slope of the area tributary to the natural buffer/filter area, and the surface slope of the natural buffer/filter area itself may not exceed 5%. In special cases steeper slopes may be used if specifically authorized by the Municipal Engineer.

c. The total impervious area tributary to a **natural** buffer / filter area can not exceed twice the buffer / filter area.

To qualify for a recharge volume credit, **constructed filter strips** shall be designed to the following standards:

a. The minimum filter strip width shall be five (5) feet or one-half of the impervious area drainage length immediately tributary to the filter strip, whichever is greater. This width is measured parallel to the direction of sheet flow.

b. The total impervious area tributary to a constructed filter strip area can not exceed twice the filter strip area.

c. The surface slope of the area tributary to the constructed filter strip area, and the surface slope of the constructed filter strip area itself may not exceed 5%

and 3% respectively. In special cases steeper slopes may be used if specifically authorized by the Municipal Engineer.

d. The filter strip surface shall consist of a minimum of six (6) inches of natural or reconstructed topsoil with a stable grass surface treatment. Reconstructed topsoil designs must be approved by the Municipal Engineer prior to application. Reconstructed topsoil consists of soils augmented by tillage and the addition of soil amendments such as compost, lime, animal manures, crop residues, etc.

e. To minimize erosion of the topsoil layer during construction, it is recommended that these areas be sodded. However, the Municipal Engineer may permit the use of an acceptable erosion control seeding application. In this later case, any loss of topsoil and seed must be replaced until a permanent vegetative stand is achieved.

3. Sidewalks separated from roadways and / or other impervious surfaces by a grass strip of equal or greater width than the sidewalk itself can be removed from the impervious area calculation when the sidewalks are graded so that sheet flow from the walk is directed to the grass strip. Sidewalks with steep longitudinal slopes that themselves would act as channels during runoff events can not take advantage of this credit. A five percent (5%) longitudinal sidewalk slope shall be used as the benchmark defining steep slopes.

4. Impervious areas tributary to natural closed depressions can be subtracted from the total site impervious area used in the recharge volume calculation as long as a qualified geotechnical engineer or soil scientist certifies to the soundness of these site specific applications. Water quality pre-treatment may be necessary prior to the direct discharge of runoff to existing closed depressions or sinkholes.

5. Impervious areas tributary to man-made closed depressions can be subtracted from the total site impervious area as long as a qualified geotechnical engineer or soil scientist certifies to the soundness of these site-specific applications. Man-

made closed depressions can be created through the use of low head berms 1 foot or less in height.

6. The entire capture volume provided in a pond without a subsurface drain may be used as a credit towards the recharge volume requirement.

7. Fifty percent (50%) of the capture volume in a pond that includes a subsurface drain may be credited towards the recharge volume requirement.

8. Additional credits may apply for undisturbed land areas that are known to have high infiltration capacity, that are maintained or enhanced. These areas must be defined and quantified from actual site data collection.

After credits, the remaining recharge volume shall be directed to a Recharge BMP such as infiltration trenches, beds, etc. These facilities can be located in open areas or under pavement structures. The appropriateness of the particular infiltration practice proposed, as well as the design parameters used, shall be supported by a geotechnical report certified by a qualified professional (soil scientist, geologist, hydrogeologist, geotechnical engineer, etc.).

Stormwater recharge requirements or credits affect stormwater management design requirements. For stormwater management computations, the reduction of site CNs based only on a weighting type analysis, as is sometimes done for cluster type developments, is not permitted. However, for stormwater management purposes, the CN for recharged areas can be computed using the NRCS method for disconnected impervious areas. The actual hydrologic process that occurs within the basin must be stressed in all recharge situations.

These recharge requirements must be met on all sites unless it can be demonstrated that recharge would be inappropriate. Any request for such a waiver from these recharge requirements must be accompanied by a supporting report certified by a qualified professional (soil scientist, geologist, hydrogeologist, geotechnical engineer, etc.).

Developers and site design professionals are encouraged to use a higher standard for recharge volume on sites where local site conditions do not restrict a higher standard.

Water Quality Sensitive (WQS) developments must use an acceptable pre-treatment BMP prior to recharge. Acceptable pre-treated BMPs for these developments include BMPs that are based on filtering, settling, or chemical reaction processes such as chemical coagulation.

Accounting for recharge within lined stormwater management ponds is not permitted. However, if unlined, uncompacted ponds and/or depressed lawn areas are used to satisfy water quality or capture volume criteria, these areas and volumes can also be used to meet recharge requirements as previously defined. Additional recharge volume may be credited to these areas as long as it is demonstrated by a qualified professional that recharge processes can naturally occur in these areas.

Finally, because this analysis is concerned with trying to adequately represent real processes that occur within the Watershed, there will be areas that cannot physically recharge stormwater. These areas include exfiltration areas that are commonly found at the base of wooded hillsides where clay pans exist, and saturation areas near major streams or floodplains. These areas may not accept recharge during most runoff events. These areas are exempt from recharge requirements when these conditions are documented and certified by a qualified professional (soil scientist, geologist, hydrogeologist, or geotechnical engineer). In addition, stormwater management techniques relying on infiltration techniques are not permitted in these areas.
The Municipal Engineer may waive the recharge requirement in the following situations:

1. The Municipal Engineer may waive the recharge requirement in highly developed areas or areas undergoing redevelopment where the Municipal Engineer has determined that forced recharge could have adverse impacts on adjacent landowner structures, property, or municipal infrastructure. These waivers should be limited to small land areas (generally less than 5 acres in size), where the ability to place recharge beds may be limited or may hinder redevelopment.

2. The Municipal Engineer may waive the recharge requirement in areas where a qualified soils scientist or geologist has determined that none of the site soils are suitable for recharge, or that the location of the suitable soils is such that harm to adjoining properties could occur as stated under item 1 above.

3. The Municipal Engineer may waive the recharge requirement in areas where recharge can not physically occur as documented by a qualified soil scientist, geologist, or hydrologist. These areas include:

    a. Exfiltration areas commonly found at the base of wooded hillsides where clay pans or fragipans exist; and

    b. Saturation areas near major streams or floodplains.

As identified above, recharge analysis and/or waiver requests must be supported by a geotechnical report sealed by a qualified professional (typically a soil scientist, geologist, hydrogeologist, or geotechnical engineer). The intent of this report will be to establish the suitability of a particular parcel of land or area for recharge, and to identify areas on a development site appropriate for recharge. It is recommended that the geotechnical / soils consultant discuss the extent and approach to the analysis with the Municipal Engineer prior to initiating the field investigation. At a minimum this report should include the following information:

1. A description of the geotechnical site investigation performed including the methods and procedures used;

2. Data presentation;

3. Analysis results including the following minimum information:

    a. A map identifying site areas inappropriate for recharge along with supporting justification. In addition to illustrating topographic features, significant

geologic and hydrologic features should be identified (rock outcrops, sinkholes, closed depressions, etc.

b. Determination of the permeability coefficient for potential recharge areas.

c. Determination of the infiltration capacity of natural site soils.

d. Location, depth, and permeability coefficient for any restrictive layers identified.

e. Soil uniformity.

f. Depth to bedrock in potential recharge areas, and a statement reflecting the uniformity of the depth to bedrock across the site.

g. A statement relating to the sites proximity to fracture zones within the bedrock.

h. Additional information deemed pertinent by the geotechnical engineer.

4. Recommendations for any special design considerations necessary for the design of recharge systems on the site. For example, required soil depth over bedrock, appropriate surface grades over recharge areas, appropriate hydraulic head over recharge areas, etc.

5. Justification as to why the site should be developed to a high impervious density if the site has adverse soil and geotechnical limitations, which prohibit the ability to induce natural recharge. Explain how these limitations will not create the potential for undue harm to the environment and the Spring Creek Watershed when the site is developed.

The following guidelines are provided relative to the use of subsurface exfiltration BMPs (often incorrectly referred to as engineered infiltration BMPs):

1. Soils should have a minimum percolation rate of 50 min/cm for effective operation of subsurface exfiltration BMPs. If no site soils have percolation rates of 50 min/cm, subsurface exfiltration BMPs should not be used.

2. A minimum of 30 inches of soil must be maintained between the bottom of a subsurface exfiltration BMP and the top of bedrock or seasonally high groundwater table. This statement is subject to the recommendation of a qualified Geotechnical Engineer.

3. If the minimum percolation rate is not met and/or the minimum soil depth can not be maintained on a site, recharge should be accommodated by directing shallow sheet flow from impervious areas across surface filter strips and/or undisturbed natural areas, or some other innovative surface infiltration feature should be used. Limiting subsurface percolation rates and/or depth to bedrock shall not by themselves warrant a recharge waiver.

In addition, since recharge is intended as a volume control, innovative or new methods that address the significant increase in the volume of runoff from sites having large impervious areas are encouraged These volume control alternatives can be used only if they can be shown to function with the original intent through sound engineering and science. The final determination of “original intent” shall always be the right of the Municipal Engineer.

D. Storm Drain Conveyance System Design

Storm drain conveyance systems consist of storm sewer pipes, swales, and open channels. The design professional should recognize that the analysis of peak runoff rate control for stormwater management is different in intent than storm drain conveyance system design. Computational methods which result in conservatively high flow estimates are desirable for conveyance design. However, if these same methods were used for peak runoff rate control (especially in the pre-development condition) the result would be frequent nuisance flooding.

Computational methods for the design of storm drain conveyance systems shall be as follows:

1. Recommended computational methods (models) for storm drain design are based on site or watershed drainage area as follows:

Up to 200 acres in size	Rational Method
Between 200 acres and 1.5 Square miles	HEC-1 PSRM TR-20
Over 1.5 square miles in size	PSU-IV with the carbonate adjustment factor at the discretion of the Municipal Engineer (refer to Appendix E).

Other methods, such as SWMM, SWIRM-ROUTE, etc., may be used if approved by the Municipal Engineer.

2. Rational Coefficients used are to be from Rawls et al. (1981), the PA DOT Design Manual 2-10, or the Aron curves used to convert CNs to C. If the Aron curves are used, all CNs must be applicable to the HSG as identified by the NRCS.

The Design Engineer may choose to use the following Rational C coefficients, without regard to soil HSG, for small sites. However, it is recommended that these coefficients be used only for storm drains up to 24 inches in diameter. The use of these conservative values shall fully be the choice of the Design Engineer.

All impervious areas: C = 0.95
All pervious areas: C = 0.30

3. Storm drains shall be designed using a 10-year storm event. Storm sewers shall be designed to pass the design flow rate without surcharging inlet structures (the hydraulic grade line elevation must remain below the ground surface at all inlets). Swales and other open channels should be designed with one (1) foot of freeboard minimum. Storm drain systems tributary to regional or multi-site

stormwater management facilities (ponds or other) must safely convey a 25-year design storm without surcharging inlets. Runoff events in excess of the indicated design event must be conveyed safely downstream.

4. Inlets on grade cannot assume a sumped condition for hydraulic modeling purposes (i.e., top of inlet casting set below pavement surface in parking areas).

5. The Municipal Engineer may require the analysis of the 100-year peak runoff rates for conveyance purposes in some instances where regional SWM facilities are employed, or to document downstream flood impacts.

6. Any storm drain that is within a State or Federal right-of-way or that falls under the design criteria of any higher authority must meet the requirements of that agency in addition to the minimum requirements of this ordinance.

7. The time of concentration (Tc) can be computed by any method that best represents the subject watershed. However, the NRCS's segmental method is not recommended for use with drainage areas that are predominately undeveloped and are greater than 100 acres in size. The NRCS Lag Equation or another more appropriate method should be used under these conditions.

8. For any drainage area smaller than 5 acres in size, a Tc of 5 minutes may always be assumed (for the post-development condition), at the discretion of the Design Engineer, without needing to provide supporting documentation.

9. Precipitation values applicable to the entire Spring Creek Drainage Basin are those reflected in PennDOT's IDF curves for Region 2, regardless if the area was formerly considered in Region 3.

10. Storm drain conveyance system stability (swales, open channels, and pipe discharge aprons) shall be computed using a 10-year return period peak runoff rate.

11. All other storm drain design methods are to be the same as specified in existing local ordinances.

12. Computational procedures other than those indicated here should follow the methods of the Federal Highway Administration's Urban Drainage Design Manual [Hydraulic Engineering Circular No 22. (HEC-22)].

E. Water Quality Standards

Water Quality Volume (WQv)

The required water quality volume that must be treated for **non-sensitive areas underlain by carbonate rock** (see exhibits 2.4 and 3.1) within the Spring Creek Basin shall be computed as:

$$WQ_{depth} = 0.25+(0.012)2.9^{(0.044(SIA))}$$

$$WQ_v = WQ_{depth}(A)/12$$

Where:
- WQ_v = water quality volume in acre-feet
- WQ_{depth} = depth in inches that must be captured for impervious areas
- SIA = percent of site impervious area (all paved areas and roof with asphalt based roofs)
- A = total of all paved areas and asphalt based roofs on site in acres

The required water quality volume that must be treated **for any WQS development, on sites in sensitive areas underlain by carbonate rock, and all areas not underlain by carbonate rock** is to be computed within the entire Spring Creek Basin as:

$$WQ_{depth} = \text{the larger of } 0.5 \text{ inches or } 0.25+(0.012)2.9^{(0.044(SIA))}$$

$$WQ_v = WQ_{depth}(A)/12$$

Where:
- WQ_v = water quality volume in acre-feet
- WQ_{depth} = depth in inches that must be captured for impervious areas
- SIA = percent of site impervious area (all paved areas and roof with asphalt based roofs)
- A = total of all paved areas and asphalt based roofs on site in acres

For designs in which the final design roof material is unknown, the Design Engineer must assume an asphalt based roof.

The water quality volume must be captured and treated through a water quality BMP over an extended period of time as per the specific requirements of each structure. The following credits are applicable to the water quality volume.

1. The entire capture volume provided in a pond without a subsurface drain may be used as a credit towards the water quality volume requirement.

2. Fifty percent (50%) of the capture volume in a pond that includes a subsurface drain may be credited towards the water quality volume requirement.

Additional credits to reduce the effective impervious area are applicable per Chapter 4.

Refer to Appendix C for examples on water quality and for differences between recharge and water quality.

Exhibit 3.1 Sensitive Areas Map

Exhibit 3.2 Stormwater Procedural Flowchart

Exhibit 3-3

Sensitive Land Areas for Well Head Protection Data Source

(Municipalities may update the sensitive area boundaries based on new research or studies as required)

Well Fields 1 and 3: Harter and Thomas Well Fields

Municipality: Harris, Ferguson, and College Townships
Well Owner: State College Water Authority
Includes wells: H7, H8, H11, H14, H22, H25
Protection Area: One-year zone of contribution
Source: Nittany Geoscience, February 1992, Figure 4

Well Field 5

Municipality: Ferguson Township
Well Owner: State College Water Authority
Includes wells: F55, F57
Protection Area: One-year zone of contribution
Source: Nittany Geoscience, February 1992, Figure 4

Well Field 6

Municipality: Benner and Patton Townships
Well Owner: State College Water Authority
Includes wells: B62, B63, B64, B65
Protection Area: One-year zone of contribution + direct upslope lands
Source: Nittany Geoscience, February 1992, Figure 4

PSU Golf Course Well Field

Municipality: Ferguson Township and the Borough of State College
Well Owner: Penn State University
Includes wells: PS28A, PS 37
Protection Area: One-year zone of contribution
Source: Nittany Geoscience, January, Figure 5

PSU Big Hollow Well Field

Municipality: Patton, Ferguson, and College Townships
Well Owner: Penn State University
Includes wells: PS2, PS14, PS16, PS17, PS24, PS26
Protection Area: One-year zone of contribution
Source: Nittany Geoscience, January, Figure 5

PSU Houserville Well Field

Municipality: Ferguson Township
Well Owner: Penn State University
Includes wells: PS33, PS 34, PS35
Protection Area: One-year zone of contribution
Source: Nittany Geoscience, January, Figure 5

Exhibit 3-3

Sensitive Land Areas for Well Head Protection Data Source (cont.)

Existing Well and Spring

Municipality: Ferguson Township
Well Owner: State College Water Authority
Includes wells: F3
Protection Area: 400' Radius + direct upslope lands

Ridgemont Wells

Municipality: Patton Township
Well Owner: Ridgemont Water Authority
Includes wells: P1, P2
Protection Area: 400' Radius

Spring Creek Park, Lemont #4, Lemont #5,and Rogers Wells, and Bathgate Springs

Municipality: College Township
Well Owner: College Township Water Authority
Includes wells: C1, C2, C3, C4, C5
Protection Area: 400' Radius

Exhibit 3-4

Soils Identified in the Centre County Soil Survey as on Flood Plains or on Terraces Above Flood Plains

Allegheny Series	Allegheny silt loam (AlB)
Atkins Series	Atkins silt loam (At)
Basher Series	Basher loam (Ba)
Chagrin Series	Chagrin Soils (Ch)
Dunning Series	Dunning silty clay loam (Du)
Lindside Series	Lindside soils (Lx)
Melvin Series	Melvin silt loam (Mm)
Monongahela Series	Monongahela silt loam (MoB)
Philo Series	Philo loam (Ph), Philo and Atkins very stony soils (Pk)
Pope Series	Pope soils (Po)
Purdy Series	Purdy silt loam (Pu)
Tyler Series	Tyler silt loam (Ty)

Chapter 4
WATER QUALITY CONSIDERATIONS

Non-point source (NPS) pollution is among the most significant environmental issue associated with urbanization and land development activities. Typical land development activities alter natural hydrologic processes by reducing or eliminating natural features that inhibit runoff such as vegetation, topsoil mass, enclosed depressions, and ponding areas. In addition, land development activity generates erosion and sedimentation problems during construction, compacts existing soils, and increases imperviousness throughout the Watershed. The altered hydrologic processes result in increased stormwater runoff (volume and peak) and reduced groundwater recharge and stream baseflows. In addition, these activities increase the transport of non-point source (NPS) pollutants to receiving waters.

Impervious surfaces accumulate a variety of urban pollutants from sources including atmospheric deposition, transportation activities, construction activities, and fertilizer applications in lawn areas. The pollutants can quickly be washed off of these source areas and delivered to downstream waters would that impact aquatic life through sedimentation and chemical contamination. The impacts of NPS pollutants on aquatic populations can be magnified by reduced baseflows and increased storm flows.

The most commonly reported pollutants associated with urban stormwater runoff include heavy metals, solids, and nutrients as described in the following:

- **Heavy Metals:** Copper, lead, and zinc are commonly found in urban stormwater runoff. Concentrations of these metals commonly exceed levels that can be toxic to aquatic life. These metals can also accumulate in sediments of streams and lakes.

- **Suspended Solids:** Suspended solids include sediments washed-off from impervious surfaces and sediments eroded from streambanks and disturbed areas such as construction sites. Both suspended and deposited sediments can have an adverse impact on aquatic life due to sediment drop-out as well as the attached transport of other pollutants.

- **Nutrients:** Urban runoff has elevated concentrations of nitrogen and phosphorus when compared to runoff from undisturbed natural areas. Excess nutrients have been identified as a major contributor to the decline in water quality within the Chesapeake Bay Watershed. Pennsylvania, along with other signatories to the Chesapeake Bay Agreement, have committed to reducing the tributary nutrient loadings by 40%.

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The following pollutants are also associated with urban stormwater discharges:

- **Organic Carbon:** Organic matter washed from impervious surfaces can create additional problems in slow moving waters or during low flow conditions. Decomposing organic matter can contribute to depleting dissolved oxygen within receiving waters.

- **Hydrocarbons:** Vehicles leak oil and grease, that contain a wide array of hydrocarbon compounds. Some of these compounds can be toxic to aquatic life in normal concentrations found in urban runoff.

- **Pesticides:** A modest number of currently used and recently banned insecticides and herbicides have been detected in urban streamflow at concentrations that approach or exceed toxic thresholds for aquatic life.

- **Chlorides:** Salts applied to road and parking surfaces during winter months appear in stormwater runoff and meltwater.

- **Thermal Impacts:** Impervious surfaces and ponded water in stormwater detention basins may increase the temperature of receiving waters and adversely impact cold water aquatic life.

- **Trash and Debris:** Considerable quantities of trash and debris can be washed along roadsides or dumped into storm drain networks and transported to receiving waters.

All of these pollutants can have significant impacts on aquatic life especially during low baseflow conditions. This is of particular concern in headwater areas of a watershed where stormwater flows can greatly exceed normal baseflow components during runoff conditions.

Water Quality Performance Standards

To minimize adverse impacts to stream health resulting from stormwater NPS pollution, standards are provided for the implementation of Water Quality Best Management Practices (BMPs) to reduce NPS pollutant loadings from land development activities. Design standards for specific water quality BMPs are available in the literature. References are suggested in the Stormwater Management Plan. However, the design of stormwater management best

management practices is an evolving science / art. Therefore, specific design parameters are not included for the following reasons:

1. The innovative design of stormwater best management practices cannot be reduced to a set of "cookbook" procedures. It requires a clear understanding of hydrologic processes within the watershed as well as for the specific site of interest. It also requires significant engineering judgment. Creating "cookbook" approaches only encourages design by unqualified individuals, and does not encourage innovative design or new ideas.

2. Current BMP designs and design parameters are based on limited data and research. This research is evolving and new standards, designs, and design parameters continue to be advanced. Including specific designs and standards in the ordinance would result in the ordinance becoming quickly outdated. To keep ordinances current, local municipalities would be required to review and update the ordinances annually.

The following performance standards and guidelines shall be addressed at all sites where stormwater management is required.

1. Site designs shall minimize the generation of stormwater runoff through the use of low-impact design techniques.

2. Stormwater runoff from all land development activities should be treated through the use of non-structural and structural BMPs to effectively treat the adverse impacts of stormwater runoff including NPS pollutants.

3. Water Quality BMPs shall be incorporated into site designs to treat the required Water Quality volume as defined in Chapter 3.

4. The desired priority of BMP selection shall be as defined in Table 4.6. The use of non-structural BMPs shall always take priority over the use of structural BMPs. The use of innovative BMPs and low-impact site planning is encouraged to reduce the generation of stormwater runoff and effectively treat pollutants transported in stormwater from the site.

5. The use of multiple non-structural water quality techniques along with new, emerging, and innovative techniques is encouraged to improve the quality of stormwater runoff to receiving areas and reduce and/or eliminate the need for structural BMPs. The Municipal Engineer should be consulted to clarify the design concept for meeting or exceeding the intent of this section

6. Where non-structural BMPs are unable to effectively treat all of the stormwater runoff generated from land development activities, structural BMPs shall be designed to capture and treat the computed water quality volume (WQ_v).

7. The priority pollutant source areas to be treated with BMPs are streets, parking lots, driveways, and roof areas.

8. Due to the karst nature of the Watershed, stormwater discharges from water quality sensitive developments (see Chapter 3) and discharges to water quality sensitive areas (see Chapter 3) will require special consideration. In these instances the applicant shall provide water quality pre-treatment (use of a filtering BMP and/or special structural design features) to prevent the discharge of stormwater contaminants to groundwater resources. In addition, hydrogeologic studies may be required to document potential karst related impacts.

9. Prior to stormwater management and water quality design, applicants should consult with the Municipal Engineer to verify stormwater quality criteria and present proposed features and concepts for the treatment of stormwater runoff. Following this meeting, the Municipal Engineer shall define any needed support studies or documentation.

Water Quality Treatment Volume (WQ_v)

Methods for computing the water quality treatment volume were documented in Chapter 3. This computed volume of stormwater runoff shall be treated in a structural BMP designed to remove NPS pollutants as efficiently as possible. The required water quality treatment volume may be reduced through the use of non-structural BMPs as outlined in Chapter 3. It is the intent of the proposed standards and criteria presented here to encourage the use of non-structural practices

as a first priority to economically and efficiently treat stormwater runoff. Structural BMPs are to be applied as a secondary priority.

Water Quality Credits

Due to the karst nature of the Spring Creek Basin, the non-structural water quality credits and techniques identified below may be limited for suitability and use based on development type and location. These limitations for use are specified in the restrictions section for each credit. The Municipal Engineer may require additional documentation or investigation prior to use of each specific credit to reduce the risks of sinkhole development or groundwater contamination for sensitive areas and development types. No area may be double counted for use with credits. The combined credits of natural area conservation and vegetated filter strips is limited to 50% of the site's impervious area. The drainage-way protection credit is limited to 50% of the site's impervious area. The maximum total water quality credit for any site may therefore be 100% of the site's impervious area.

Non-Structural Technique	Water Quality Credit
Drainage-way Protection (DWP)	Subtract Drainageway Protection Areas from impervious site area in WQ_v computation.
Natural Area Conservation (NAC)	Subtract Conserved Natural Areas from impervious site area in WQ_v computation.
Filter / Buffer Area	Subtract impervious areas discharged over pervious areas from impervious site area in WQ_v computation.

Drainage-way Protection (DWP)

A water quality credit is given for the protection of natural drainage-ways on a development site. Natural karst drainage-ways within the Spring Creek Watershed often do not exhibit a defined channel bed and banks. More often, these drainage-ways appear as wide, shallow parabolic swales. These drainage-ways are an integral part of the natural drainage system, and often exhibit significant infiltration capacity. Protection of these drainage-ways is critically important to health of the watershed.

The drainage-way protection (DWP) area is defined as an area centered on the drainage-way and having a maximum width of 300 feet and a minimum width of 50 feet. The Municipal Engineer may modify the defined minimum width in cases where natural land forms define an appropriate alternate width or for small sites.

The impervious area used in the WQ_v equation for the development site may be reduced by **twice** the area of the preserved drainage-way (2 to 1 ratio).

1. Restrictions on the Credit:

   - Drainage-way protection areas must remain in an undisturbed condition during and after construction activities. There can be no construction activity within these areas including temporary access roads or storage of equipment or materials. Temporary access for the construction of utilities crossing this protection area may be permitted at the Municipal Engineer's discretion. However, the alignment of any such crossing must be perpendicular to the drainage-way.

   - These areas should be placed in a conservation easement or permanently preserved through a similarly enforceable agreement with the municipality.

   - The limits of the undisturbed DWP area and conservation easement must be shown on all construction plans.

   - The DWP area must be located on the development site.

   - The maximum total DWP credit is 50% of the site impervious area.

   - Water quality credits are not permitted for Water Quality Sensitive (WQS) developments.

2. Sensitive Area and Water Quality Sensitive Development Restrictions:

   - DWP areas may not be counted as a credit in sensitive areas unless the impervious area actually flows across the area as sheet flow.

- Untreated urban runoff from water quality sensitive developments (WQS) may not be directed to DWP areas without pretreatment.

Natural Area Conservation (NAC)

A water quality credit is given for natural areas that are conserved at the development site, thereby maintaining pre-development water quality characteristics. The impervious area used in the WQ_v equation for the development site may be reduced by the natural area conserved in the water quality volume computations. Natural area conservation (NAC) is different than vegetated filter strip/recharge area and drainageway protection in that in some cases surface runoff may never be directed over the natural area (i.e., if upslope wooded areas are conserved).

1. Restrictions on the Credit:

- Natural areas must remain in an undisturbed condition during and after construction activities. Temporary incidental land disturbance activities associated with utility construction may be permitted within the conservation area.

- These areas should be placed in a conservation easement or similarly enforceable agreement with the municipality.

- The limits of the undisturbed area and conservation easement must be shown on all construction plans.

- The area must be located on the development site.

- Water quality credits are not permitted for Water Quality Sensitive (WQS) developments.

- The maximum total NAC credit is 50% of the site impervious area. However, the combination of NAC filter buffer area credit is also 50%.

2. Sensitive Area and Water Quality Sensitive Development Restrictions:

- NAC areas may not be counted as a credit in sensitive areas unless the impervious area actually flows across the area as sheet flow.

- Untreated urban runoff from water quality sensitive developments (WQS) types may not be directed to natural areas without pretreatment.

Filter / Buffer Area

A water quality credit is given when stormwater runoff is effectively treated via a filter / buffer area or strip. A filter / buffer area is a vegetated boundary characterized by uniform mild slopes. Filter strips may be forested or vegetated with turf grass. Effective treatment is achieved when impervious area runoff is directed as sheet flow across vegetative filter or buffer areas (i.e., concentrated flow discharged to a filter strip does not meet water quality reduction criteria).

The area draining via overland sheet flow to an undisturbed, natural, vegetated filter strip (natural unmaintained meadow or forested area) can be subtracted from the site impervious area (IA) on a 1:1 area ratio in the water quality volume computation. Impervious areas draining across constructed (disturbed or regarded) pervious areas can be subtracted from the site impervious area (IA) on a 1:1/2 area ratio in the water quality volume computation.

1. Restrictions on the Credit:

- The maximum impervious area that can be included in this credit, shall be computed as follows:

$$IA_c = W_{IA} \; L_{IA}$$

Where:
IA_c = Impervious area recharge credit (L^2).
L_{IA} = Length of impervious area measured perpendicular to the sheet flow direction (L).
W_{IA} = Width of impervious area (L). Maximum width permitted for credit is the smaller of 100 feet or twice the width of the vegetated filter strip.

- To qualify for a water quality credit, natural and constructed filter areas or strips must meet the same restrictions identified for natural or constructed recharge

areas with regard to width, length, slope, tributary drainage length, and construction. These restrictions are presented in Chapter 3.

- Runoff shall enter the filter / buffer strip as overland sheet flow.

- Filter/ buffer areas shall remain undisturbed/unmanaged other than to remove accumulated trash and debris.

- Water quality credits are not permitted for Water Quality Sensitive (WQS) developments.

- The maximum total water quality credit for vegetative filter / buffer areas is 50% of the site impervious area. However, the combination of NAC and filter / buffer areas is also 50%.

2. Sensitive Area and Water Quality Sensitive Development Restrictions:

- NAC areas may not be counted as a credit in sensitive areas unless the impervious area actually flows across the area as sheet flow.

- Untreated urban runoff from Water Quality Sensitive (WQS) developments may not be directed to filter / buffer areas without pretreatment.

Comments Related to Water Quality Credits

Concurrence of the Municipal Engineer is required prior to the use of all water quality credits for the reduction of the water quality treatment volume. The Municipal Engineer may approve the use of additional credits based upon sufficient documentation regarding suitability for sensitive development types and areas, pollutant removal effectiveness, and maintenance criteria. Multiple water quality credits cannot be claimed for the identical area of the site (i.e., a stream buffer credit and disconnecting roof recharge area cannot both be claimed for the same area).

Additional impervious coverage reduction using low impact development techniques (development practices which reduce the impact of urban runoff such as narrower residential road sections, smaller cul-de-sacs, smaller parking stalls, smaller building set-backs to reduce driveway lengths, etc.) will also reduce the required water quality treatment volume. Many of these techniques require prior approval by the municipality before implementation into land development design.

Best Management Practices

Water Quality Best Management Practices (BMPs) include the use of structural and non-structural techniques and devices to reduce or prevent the transport of NPS pollutants to receiving creeks, streams, and other bodies of water. Structural BMPs can be categorized as follows **(there is no implied priority of effectiveness in the following lists)**:

- stormwater ponds;
- stormwater wetlands;
- recharge practices;
- filtering practices; and
- open channel practices.
- water quality inlets

Structural BMPs , even BMPs such as porous pavements or engineered bioinfiltration areas, should always be considered less beneficial than stable natural treatment areas.

Non-structural techniques include but are not limited to:

- natural area conservation (including conservation of forested riparian buffers);
- disconnection of rooftop and other impervious areas;
- sheet flow discharge to stream buffers;
- filter strips;
- use of grass lined open channels for conveyance;
- environmentally sensitive or low impact rural development; and
- impervious cover reduction.

The following sections provide guidelines for the selection of BMPs within the Spring Creek Watershed. Specific BMPs within each of the categories identified above are further defined. A process for selecting the best BMP or combination of BMPs is then presented based on stormwater treatment suitability, terrain, physical and karst limitations, and community and environmental factors.

A wide variety of BMP designs are currently being promoted in various state and other publications including:

- "The Pennsylvania Handbook of Best Management Practices for Developing Areas" (CH2Mhill, 1997);
- "Low Impact Development Design Manual", Prince George's County, Maryland;
- "Design Manual for Use of Bio-retention in Stormwater Management", (Engineering Technologies Associates, Inc. for The Department of Environmental Resources, Division of Environmental Management, Watershed Protection Branch, Prince George's County, Maryland, 1993); and
- "The Maryland Stormwater Design Manual" (Center for Watershed Protection, Inc., et.al., 1997).

The technical criteria, standards, and details provided in these documents should be used as a basis for the design of BMPs. Innovative and new BMP designs are encouraged. However, it is recommended that the design standards and procedures proposed for use on a particular site be reviewed with and approved by the local municipal authority prior to submission of preliminary or final plans for review. It is suggested that these procedures be reviewed with the municipal authority as a part of sketch plan discussions.

Typical BMPs within each of the functional categories identified above are discussed in the following sections.

Group 1. Stormwater Ponds

Typical stormwater pond practices include:

- Dry Extended Detention Ponds
- Wet Ponds
- Wet Extended Detention Ponds

These practices are typically used for stormwater quantity control. However, they do provide some reduction in TSS from a water quality standpoint, particularly when combined with a extended detention feature and/or forebay features. A typical extended detention pond is illustrated in Figure 4.1. Wet ponds have a maintained permanent pool elevation, as illustrated in Figure 4.2. An extended detention feature can also be added to wet ponds.

Group 2. Stormwater Wetlands

Stormwater wetlands are structures that include significant shallow marsh areas used to treat urban stormwater, and they often may also incorporate small permanent pools and/or extended detention storage to achieve the desired water quality effect. These structures include:

- Shallow Wetlands
- Pond/Wetland Combinations

A shallow wetland is illustrated in Figure 4.3. By adding a deeper and more extensive permanent pool area, a pond/wetland is achieved, as illustrated in Figure 4.4. Extended detention control can be added to either of these structures to enhance pollutant removal.

Group 3. Recharge Practices

Recharge practices capture, temporarily store, and then allow stormwater to infiltrate into the soil over an extended period of time. These practices include:

- Infiltration Trenches or Beds
- Infiltration Basins

Infiltration practices have an exceptional water quality mitigation value. Figure 4.5 illustrates a typical infiltration trench. The width of an infiltration trench can be exaggerated to create an

infiltration bed in areas where additional infiltration is required or where ponding depth is a concern. An infiltration basin is illustrated in Figure 4.6.

Group 4. Filtering Practices

Filtering practices also provide exceptional water quality mitigation value. Filtering practices capture, temporarily store, and then pass the water quality volume through a filter bed of sand, organic matter, soil, or other media. These practices include:

- Sand or organic filters;
- Bio-retention areas;
- Use of stable natural closed depressions;
- Use of engineered low head depressions.

Most filters are constructed as under-drained permeable trenches as illustrated in Figure 4.7. Although extremely expensive, large manufactured tanks have also been used to house filter materials (Figure 4.8). Bio-retention areas are similar to standard filters with the addition of mulch and surface plant materials to enhance biological uptake. Figure 4.9 illustrates a typical bio-retention facility.

Group 5. Open Channel Practices

This practice includes open channels that are explicitly designed to capture and treat all or part of the design water quality volume. These features are referred to as:

- Water Quality Swales (dry or wet)

Figure 4.10 illustrates a water quality swale. Water quality channels or swales differ from simple grass lined swales in that they are constructed on very mild slopes and often include check dams or other means to create wet cells and encourage infiltration. If soils do not provide sufficient infiltration capacity, a filter and underdrain can be included in the swale.

Group 6. Non-Structural Practices

As described in the Maryland Stormwater Design Manual, non-structural BMPs are increasingly recognized as a critical feature of stormwater BMP plans, particularly with respect to site design. In most cases, non-structural BMPs are combined with structural BMPs to meet all stormwater requirements. The key benefit of nonstructural BMPs is that they can reduce the generation of stormwater from the site, thereby reducing the size and cost of structural BMPs. In addition, they can provide partial removal of many pollutants. Typical non-structural BMPs include:

- Natural Area Conservation (including forested riparian buffer conservation and conservation of natural karst drainage-ways);
- Disconnection of Rooftop and Other Impervious Areas;
- Sheet Flow Discharge to Stream Buffers;
- Filter Strips;
- Use of Grass-lined Open Channels for Conveyance;
- Environmentally Sensitive or Low Impact Rural Development; and
- Impervious Cover Reduction.

Group 7. Water Quality Inlets

Several innovative water quality inlet structures have been recently developed and information can be obtained from manufacturers or suppliers. When these structures are used they should be designed and installed per the manufacturer's specifications. For this reason, water quality inlets are not covered further in this chapter, but are a permitted use.

BMP Selection Criteria

This section outlines a process for selecting an appropriate BMP or group of BMPs for a development site. The information provided here is the end product of a study to identify selection criteria for BMPs within the Spring Creek Watershed (Sweetland Engineering & Associates, Inc., 2000). The selection process is qualitative in nature and includes application of screening criteria and consideration of a BMP selection priority matrix. The selection criteria are based on an understanding of hydrologic processes within the Spring Creek Watershed, the

results of the Shiloh Road test basin analysis conducted as a part of the aforementioned study, and a review of related literature.

In the following sections, screening criteria and a prioritization matrix are presented as tools for BMP selection. Criteria for the use of multiple non-structural techniques are also presented. These tools are intended to provide the designer with a means of assessing the relative effectiveness and applicability of BMPs within the Spring Creek Watershed. However, structural BMPs, even BMPs such as porous pavements or engineered bioinfiltration areas, should always be considered less beneficial than stable natural treatment areas.

BMP Screening Criteria

The selection of an appropriate BMP relies on a variety of site, watershed, and community factors including treatment suitability, terrain, physical features, and community and environmental factors. It is the interaction of these factors that defines the most appropriate BMPs for a particular site and development activity. These criteria are described in the following sections.

Stormwater Treatment Suitability

Stormwater treatment suitability considers the applicability of a specific BMP for mitigation of NPS pollutant discharges. BMP suitability requires that the BMP's ability to mitigate NPS pollutant loads generally matches or correlates with the pollutant loadings being generated.

BMP removal efficiencies for individual practices can vary widely. Tables 4.1 and 4.2 were created to provide a more generalized ranking of pollutant loadings and removal efficiencies, respectively. These tables are used as the foundation upon which the BMP prioritization matrix presented in the next section is based.

Several observations can be made from the data reported in Tables 4.1 and 4.2. First, there is only minor variation in the increased level of pollutant loading among the common classifications of pollutants found in urban runoff (metals, solids, and nutrients) for a given land

Table 4.1
Levels of Increased Pollutant Loading as a Function of Land Use

Land Use	Heavy Metals			Solids	Nutrients	
	Lead (Pb)	Copper (Cu)	Zinc (Zn)	TSS	Nitrogen (TKN)	Phosphorus (TP)
Commercial	**H**	**H**	**H**	**H**	**H**	**H**
Parking Lot	**M**	**L**	**M**	**M**	**H**	**M**
High Density Residential	**M**	**L**	**M**	**M**	**M**	**M**
Medium Density Residential	**L**	**M**	**M**	**M**	**M**	**M**
Low Density Residential	**L**	**L**	**L**	**L**	**L**	**L**
Freeway	**H**	**EX**	**H**	**H**	**H**	**M**
Industrial	**H**	**EX**	**EX**	**H**	**M**	**H**
Park / Maintained Open	**L**	**L**	**L**	**L**	**L**	**L**
Open, Unmaintained	**L**	**L**	**L**	**L**	**L**	**L**

* Table Based in Table 4-3 Preliminary Data Summary of Urban Stormwater Best Management Practices (US EPA 1999)

Key:

Extreme increase in pollutant loading	**EX**	Moderate increase in pollutant loading	**M**
High Increase in pollutant loading	**H**	Low increase in pollutant loading	**L**

Table 4.2
BMP Pollutant Removal Efficiency

Best Management Practice	Heavy Metals			Solids	Nutrients	
	Lead (Pb)	Copper (Cu)	Zinc (Zn)	TSS	Nitrogen (TKN)	Phosphorus (TP)
Ponds						
Extended Detention Dry	**L**	**L**	**L**	**L**	**L**	**L**
Wet	**H**	**M**	**M**	**H**	**L**	**M**
Extended Detention Wet	**M**	**M**	**M**	**H**	**L**	**M**
Wetlands						
Shallow Marsh	**M**	**M**	**M**	**H**	**M**	**M**
Pond / Wetland Combination	**H**	**M**	**M**	**H**	**L**	**M**
Recharge						
Infiltration Trench or Bed	**HH**	**HH**	**HH**	**HH**	**H**	**H**
Infiltration Basin	**HH**	**HH**	**HH**	**HH**	**H**	**H**
Filters						
Sand or Organic	**H**	**L**	**H**	**HH**	**M**	**M**
Bioretention	**HH**	**HH**	**HH**	**H**	**H**	**H**
Open Channel						
Water Quality Swales	**H**	**M**	**H**	**HH**	**L**	**L**
Non-Structural						
Grassed Channels	**M**	**M**	**M**	**H**	**L**	**L**
Filter Strips	**M**	**M**	**M**	**H**	**L**	**L**

Key:

Highest (>80%)	**HH**	Moderate (40% to 60%)	**M**
High (60% - 80%)	**H**	Low	**L**

use. While there are some significant differences in the increased pollutant loading associated with different land uses, within a given land use; there is only minor variation among the pollutant classifications. This is demonstrated by the data presented in Table 4.1. The second observation is that BMPs that have high pollutant removal efficiencies are effective across the spectrum of pollutants. Table 4.2 illustrates this observation. While there are some exceptions, BMPs that are effective at removing heavy metals, are also good at removing solids and nutrients.

Based on the information presented in Tables 4.1 and 4.2, the following generalized conclusions can be made regarding the application of BMPs for specific land uses:

1. Commercial, freeway, and industrial zoning districts produce the highest levels of pollutants. Urban stormwater from these land use areas should be treated with a high efficiency BMP such as recharge practices (infiltration beds, trenches, and basins) or filtering practices (bioretention, or a good sand or organic filter).

2. Parking lots, high density residential (>50% imperviousness), and medium density residential (30% to 50% imperviousness) zoning districts produce consistently moderate levels of urban pollutants. These land use areas can effectively be treated with wet ponds, wetlands, recharge, or filtering practices.

3. Grass channels and filter strips are best suited for removal of heavy metals and suspended solids in medium (<50% imperviousness) or lower density developments. They are not effective for nutrient removal.

4. Water quality swales, due to their high removal efficiency for heavy metals and suspended solids, are applicable for the full range of development densities. However, they are not effective for nitrogen and phosphorus removal.

The stormwater pollutant potential from land uses identified in Items 1 and 2 above can be quite high. In these areas, recharge must be accomplished carefully, with consideration given to the proper arrangement of treatment and recharge functions. Adequate treatment must be accomplished prior to direct recharge to groundwater sources. In addition, treatment BMPs

should be designed in such a way that their performance can be checked, and they can be isolated, repaired, or re-configured in the event they do not function as designed.

Terrain Factors

The two (2) major terrain factors considered within the Spring Creek Watershed are karst and mountain terrain. These factors are presented in Table 4.3. A large portion of the Spring Creek Watershed is underlain with carbonate rock. Karst features consisting of closed depressions, sinkholes, solution channels, and caverns predominate the valley region. This region poses special limitations relative to the implementation of BMPs, particularly recharge practices.

The major karst concern is the introduction of direct surface runoff to ground water via sinkhole creation. Many of the sinkholes that develop within the Spring Creek Watershed develop in areas where surface flows have been redirected due to development or in areas where stormwater has been ponded for detention purposes. Therefore, two (2) of the most important karst factors are the maintenance of existing drainage patterns, when possible, and the use of low head BMP designs. The relative risk for sinkhole development associated with each BMP group is identified in Table 4.3.

Table 4.3
Terrain Factors

BMP Group	Karst Factors	Sinkhole Risk	Mountainous
Ponds	• A synthetic or clay liner may be required. • Wet ponds will require supporting geotechnical investigation and calculations. • Maximum permanent pool – 3'. • Maximum temporary high water – 8'. • Minimum 4' separation between pond bottom and rock.	• Higher Risk	• Embankment heights restricted. • Maximum permanent pool – 6'. • Maximum temporary high water – 10'.
Wetlands	• Supporting geotechnical investigation required. • A synthetic or clay liner may be required. • Minimum 4' separation between wetland bottom and rock. • Maximum permanent pool – 3'. • Maximum Temporary high water – 8'.	• Higher Risk	• Embankment height restricted. • Maximum permanent pool – 6'. • Maximum temporary high water – 10'.
Recharge	• Supporting geotechnical investigation required. • Maximum head in infiltration practice to invert of relief pipe 18". Greater heads can be permitted if supported by geotechnical report. • Minimum 2' to water table. • May be prohibited if the contributing drainage area land use is considered a significant pollutant source.	• Lower Risk if hydraulic head limited.	• Max slope – 6% • Trenches and beds must have flat bottoms
Filtering	• Synthetic or clay liner required unless lack of liner is supported by a geotechnical investigation. • Unlined filter systems may have head restrictions.	• Lower Risk	• OK
Open Channels	• Maximum head limited to 18" unless supported by a geotechnical investigation.	• Lower Risk	• Not applicable if slopes are greater than 4%.

Physical Features Factors

Physical conditions or features at the site often pose limitations related to the application of specific BMPs. The four primary factors within the Spring Creek Watershed are:

- ***Water Table:*** This column indicates the minimum depth to the seasonally high water table from the bottom or floor of a BMP.
- ***Drainage Area:*** This column indicates the minimum or maximum drainage area that is considered suitable for the practice. If the drainage area present at a site is slightly greater than the maximum allowable drainage area for a practice, some leeway is permitted. The minimum drainage areas indicated for ponds and wetlands should not be considered inflexible limits and may be increased or decreased depending on water availability (base flow or groundwater) or the mechanisms employed to prevent clogging.
- ***Site Slope:*** This column evaluates the effect of slope on the practice. Specifically, the slope restrictions refer to how flat the area where the practice is installed must be and how steep the contributing drainage area or flow length can be.
- ***Head:*** This column provides an estimate of the minimum elevation difference needed at a site (from the inflow to the outflow) to allow for gravity operation within the practice.

Physical feasibility factors are identified in Table 4.4 found on the next page.

Community and Environmental Factors

Identified Community and Environmental Factors include ease of maintenance, community acceptance, affordability, and habitat. These factors are described below. The matrix illustrating these factors is illustrated in Figure 4.5 and employs a comparative index approach. A dark circle indicates that the BMP has a high benefit, and an open circle indicates that the particular BMP has a low benefit.

- ***Maintenance:*** This column assesses the relative maintenance effort needed for a BMP, in terms of three criteria: 1) frequency of scheduled maintenance; 2) chronic maintenance problems (such as clogging); and 3) reported failure rates. It should be noted that **all BMPs** require routine inspection and maintenance.
- ***Community Acceptance:*** This column assesses community acceptance, as measured by three factors: 1) market and preference surveys; 2) reported nuisance problems; and

3) visual orientation (i.e., is it prominently located or is it in a discrete underground location). It should be noted that a low rank can often be improved by a better landscaping plan.

- ***Affordability:*** The BMPs are ranked according to their relative construction cost per impervious acre treated as determined from cost surveys and experience.

- ***Habitat:*** BMPs are evaluated on their ability to provide wildlife or wetland habitat, assuming that an effort is made to landscape them appropriately. Objective criteria include size, water features, wetland features, and vegetative cover of the BMP and its buffer.

- ***Other Factors:*** This column indicates other considerations in BMP selection.

Table 4.4
Physical Feasibility Factors

BMP List	Water Table	Drainage Area (acres)	Site Slope	Head (ft)
Ponds				
Extended Detention Dry		No min		
Wet		25 min*		
Extended Detention Wet				
Wetlands	Two (2) feet if infiltration processes are required otherwise zero (0) feet.	25 min*	No more than 8%	3 to 5 feet
Shallow Marsh				
Pond / Wetland Combination				
Infiltration	Three (3) feet		No more than 6%	
Trench or Bed		5 max		2 feet
Basin		10 max		3 feet
Filters	Two (2) feet	5 max; 10 max for surface sand filter**	No more than 6%	
Sand or Organic				2 to 4 feet
Bioretention				5 feet
Open Channel	Two (2) feet	5 max	No more than 4%	2 feet
Water Quality Swales				
Non-Structural	Two (2) feet			
Grassed Channels			No more than 4%	3 feet
Filter Strips			No more than 2% on filter	2 – 3 inches

Notes: * Unless adequate water balance and anti-clogging device installed.
**** Drainage area can be larger in some instances.**

Table 4.5
Community and Environmental Factors

BMP List	Ease of Maintenance	Community Acceptance	Affordability	Habitat	Other Factors
Ponds					
Extended Detention Dry	●	●	◗	❍	
Wet	●	◗	◗	●	High Pond Premium
Extended Detention Wet	●	◗	◗	●	
Wetlands					
Shallow Marsh	◗	●	◗	●	
Pond / Wetland Comb.	●	●	◗	●	
Infiltration					
Trench or Bed	❍	●	❍	❍	Avoid Large Stone
Basin	❍	❍	◗	❍	Frequent pooling
Filters					
Sand or Organic	◗	●	❍	❍	Change Compost
Bioretention	❍	◗	❍	❍	Landscaping
Open Channel					
Water Quality Swales	●	●	●	❍	
Non-Structural					
Grassed Channels	●	●	●	❍	
Filter Strips	●	●	●	❍	
Benefits	● High	◗ Medium	❍ Low		

BMP Selection Priority Matrix

To assist in the selection of an appropriate BMP for a specific site, a prioritization matrix was developed (Sweetland Engineering & Associates, Inc., 2000). The following elements from the screening matrices were used as a basis for developing a prioritized ranking of BMP types:

- Treatment suitability;
- Relative risk of sinkhole development; and
- Relative cost.

These factors were chosen as a reasonable approach to identifying preferred BMPs suitable within a karst environment. Each BMP or BMP group was given a numerical rank to assess its overall suitability within the watershed with regard to each of the three factors identified above.

Table 4.6 documents the prioritization of preferred BMPs and the scores for heavy metals, solids, and nutrients. This table indicates that, with the exception of one value for solids and one value for nutrients, the preferred BMP prioritized ranking is the same for heavy metals, solids, and nutrients. Although terrain factors such as site slope and depth to water table impact the suitability of each measure on a case-by-case basis, the general prioritization is useful for the designer to demonstrate to the reviewing municipality that due thought has been used in the selection of BMPs for each plan.

Table 4.6
BMP Prioritization Matrix

Best Management Practice	Heavy Metals Score	Solids Score	Nutrients Score
Water Quality Swales	8.7	10.0	7.0
Filter Strips	8.0	9.0	7.0
Recharge	8.0	8.0	7.0
Bioretention	8.0	7.0	7.0
Shallow Marsh	7.0	8.0	7.0
Sand or Organic Filters	6.3	8.0	6.0
Wetland Creation	5.3	6.0	4.5
Pond / Wetland Combination	5.3	6.0	4.5
Extended Detention Dry	5.0	6.0	4.5
Extended Detention Wet	5.0	5.0	5.0

Note that the matrix presented in Figure 4.6 is intended as a planning tool for municipal officials, designers, and developers during the planning process for land development projects. However, this matrix should not be considered alone. Other factors, included in the screening matrices and site specific considerations must also be considered when developing a stormwater quality design.

The design and implementation of new and innovative BMPs not included in this analysis is encouraged if it can be reasonably demonstrated that they have a pollutant removal capability

that is acceptable to the municipalities. In addition, adequate design measures should always be taken to reduce the risk of sinkhole formation.

Recommendations for Application of BMP Selection Criteria

Based on the selection criteria presented above, the following recommendations are made for BMP selection within the Spring Creek Watershed:

1. Non-structural techniques should be used to the maximum extent possible where appropriate. Non-structural techniques include:

    - Natural Area Conservation (including forested riparian buffer conservation and conservation of natural karst drainage-ways);
    - Disconnection of Rooftop and Other Impervious Areas;
    - Sheet Flow Discharge to Stream Buffers;
    - Use of Grass Lined Open Channels for Conveyance;
    - Environmentally Sensitive Rural Development; and
    - Impervious Cover Reduction.

2. Treating the runoff from impervious surfaces should be the highest priority for land development projects.

3. Information in the screening and prioritization matrices should be used as a basis for the selection of best management practices for water quality control. Creative designs implementing a variety of control measures are encouraged. The following considerations should be made during the design process:

    - Use the pollutant loading and pollutant removal efficiency matrices (Tables 4.1 and 4.2 respectively) to identify the types of pollutants that may be of concern for the particular development and BMPs that are effective at removing those pollutants.

- Identify other constraints in Tables 4.3, 4.4, and 4.5 that may limit the use of specific BMPs or BMP groups.

- Use the BMP Prioritization Matrix (Table 4.6) to select the BMP or group of BMPs that are ranked the highest given the other constraints identified.

- Initiate the BMP selection process during conceptual site layout and design to take maximum advantage of natural site characteristics and minimize costs associated with the construction of structural BMPs.

Following the recommendations outlined above will provide for reasonable selection and design of appropriate BMPs within the Spring Creek Watershed. While there are a number of factors influencing the design and selection process, it should be the intent of the designer to utilize existing and emerging BMP technologies to minimize the impact of NPS pollutants on the surface and ground water resources within the Watershed.

Recommended BMP Design Criteria

Technical standards for the design of BMPs are currently available in a number of publications prepared by both private and public organizations. The following list of publications are recommended as a source of technical design standards for BMP design:

- "The Pennsylvania Handbook of Best Management Practices for Developing Areas" (CH2Mhill, 1997);
- "Low Impact Development Design Manual", Prince George's County, Maryland;
- "Design Manual for Use of Bio-retention in Stormwater Management", (Engineering Technologies Associates, Inc. for The Department of Environmental Resources, Division of Environmental Management, Watershed Protection Branch, Prince George's County, Maryland, 1993);
- "The Maryland Stormwater Design Manual" (Center for Watershed Protection, Inc., et.al., 1997).

In addition, recommended design criteria for vegetated open channels considered as water quality BMPs are provided in the next section.

Other sources for technical design standards that may become available from time to time may also be used. However, the technical standards used must be referenced in the Stormwater report. If requested, the source documentation must be provided to the Municipal Engineer or reviewing authority.

Recommended Design Criteria for Vegetated Open Channels Considered as Water Quality BMPs

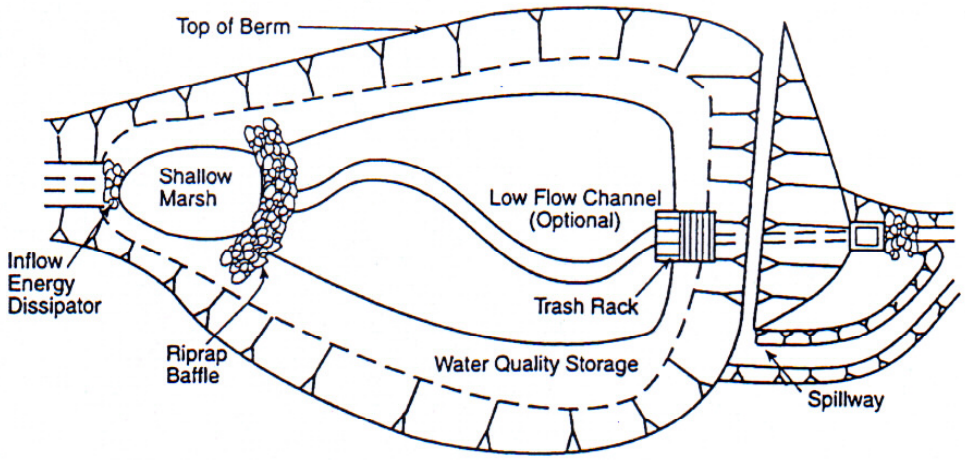
If a vegetated open channel is designed and constructed within the following restrictions and limitations, the tributary impervious area shall be considered to meet water quality goals.

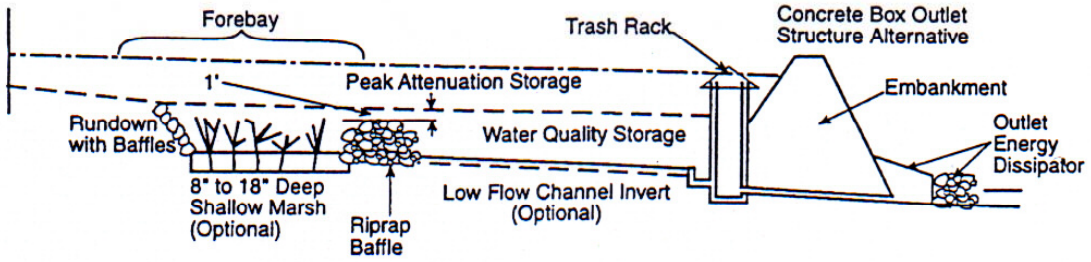
1. Restrictions for Use:

   - The maximum flow velocity for runoff from the water quality depth shall be less than 1.0 fps.
   - The minimum bottom width of the channel shall not be less than 2.0 feet.
   - The side slopes shall be 2:1 or flatter.
   - The average channel slope shall be less than 4.0%.
   - The channel must be documented at the concept design phase and verified after final grading.
   - Small water quality check dams may be placed in the channels to increase their effectiveness.

2. Sensitive Area and Water Quality Sensitive Development Restrictions:

   - Structural BMP pretreatment of urban runoff and/or channel lining may be required prior to discharge of runoff from sensitive development types into vegetated open channels.





(Adapted from *Dam Design and Construction Standards*, Fairfax County, Virginia, 1991)

Figure 4.1
Extended Detention Pond

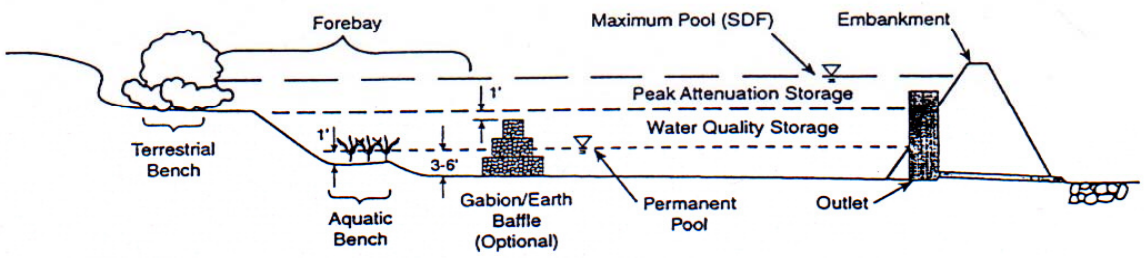
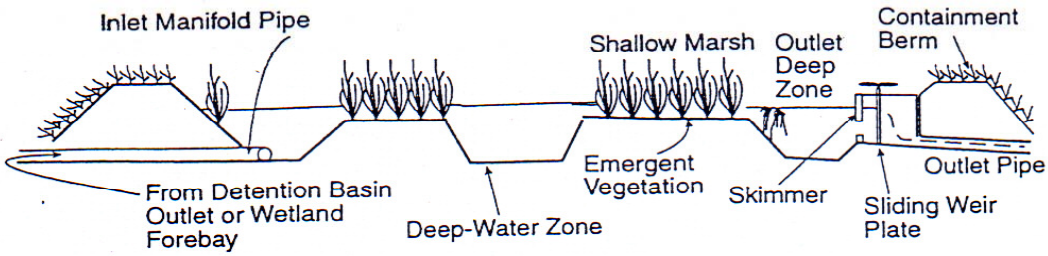


Figure 4.2
Wet Pond



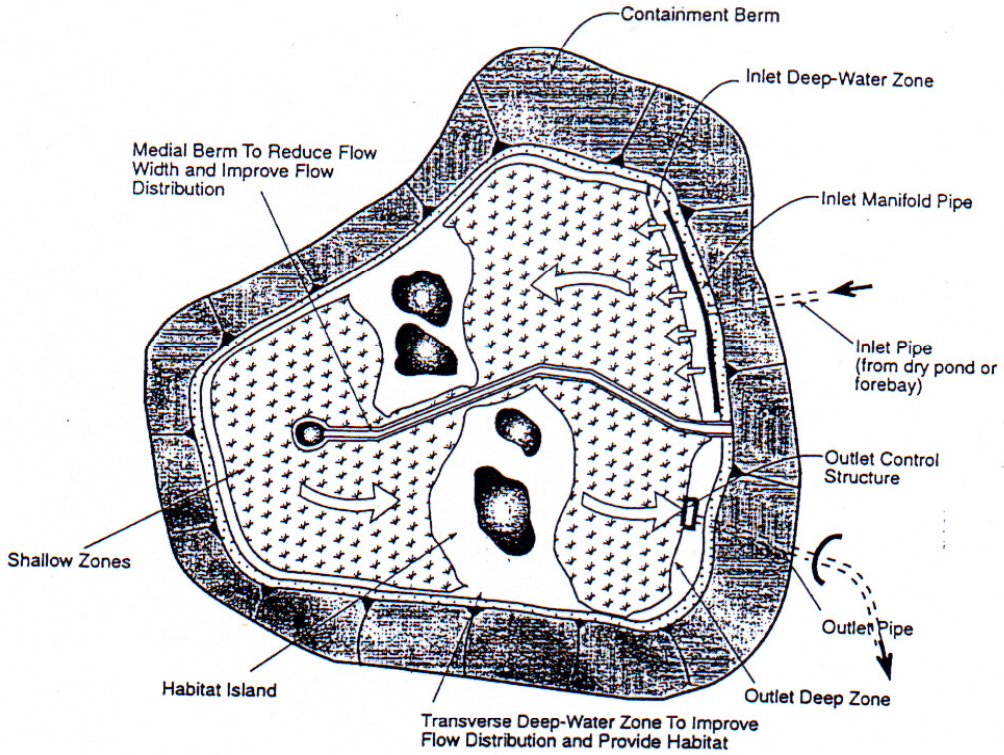


Figure 4.3
Shallow Wetland

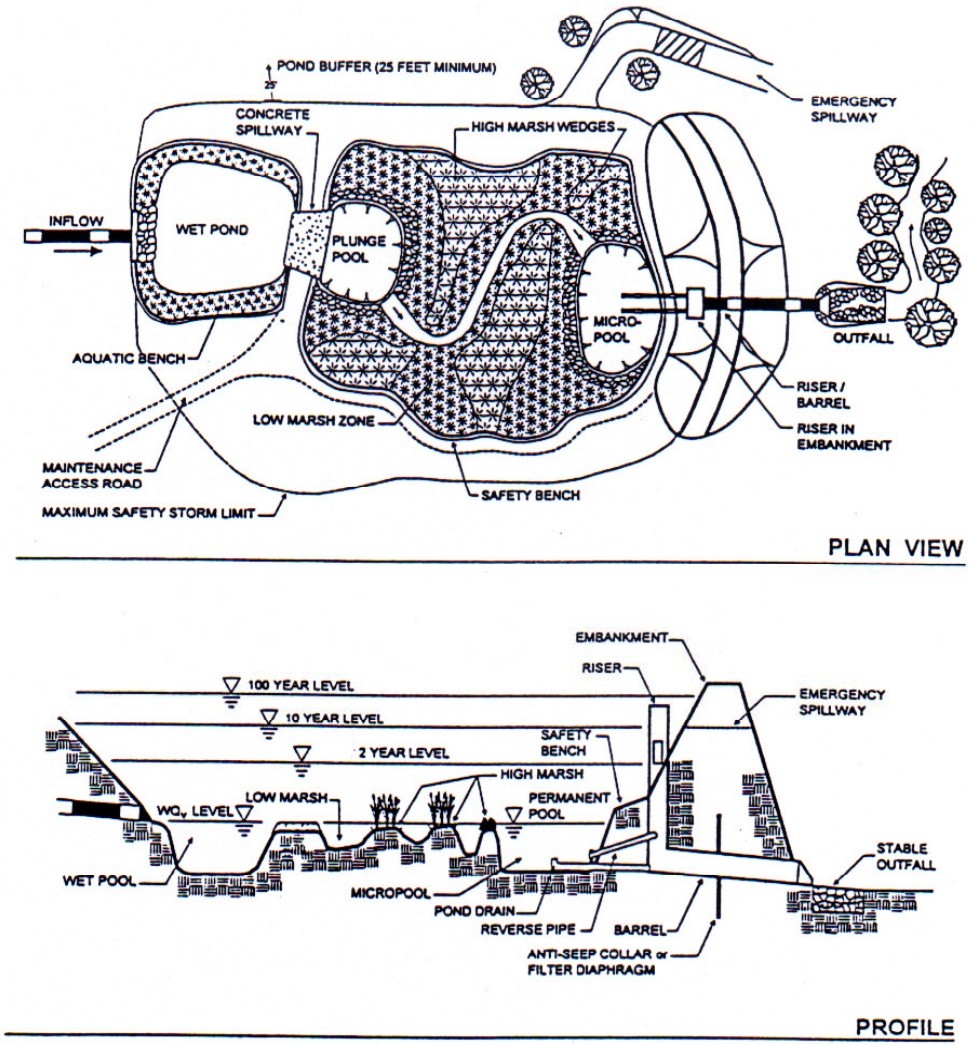
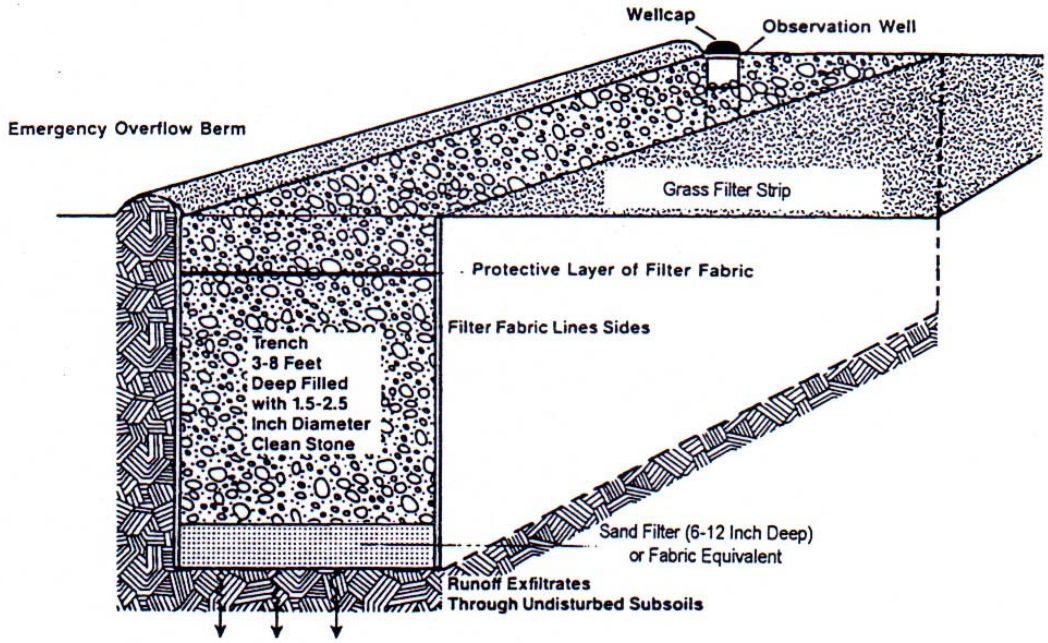


Figure 4.4
Wetland / Pond Combination



(Adopted from Schueler *et al.*, 1992)

Figure 4.5
Infiltration Trench

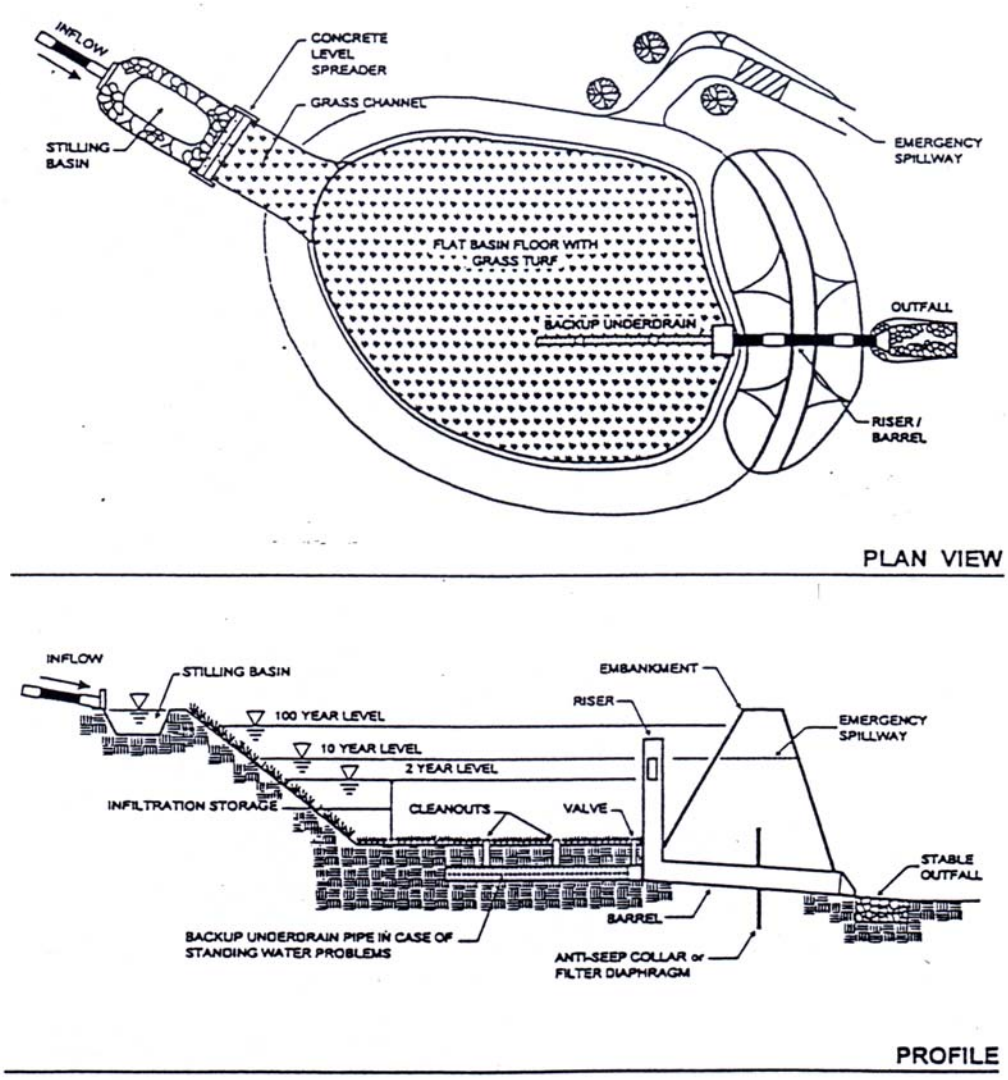


Figure 4.6
Infiltration Basin

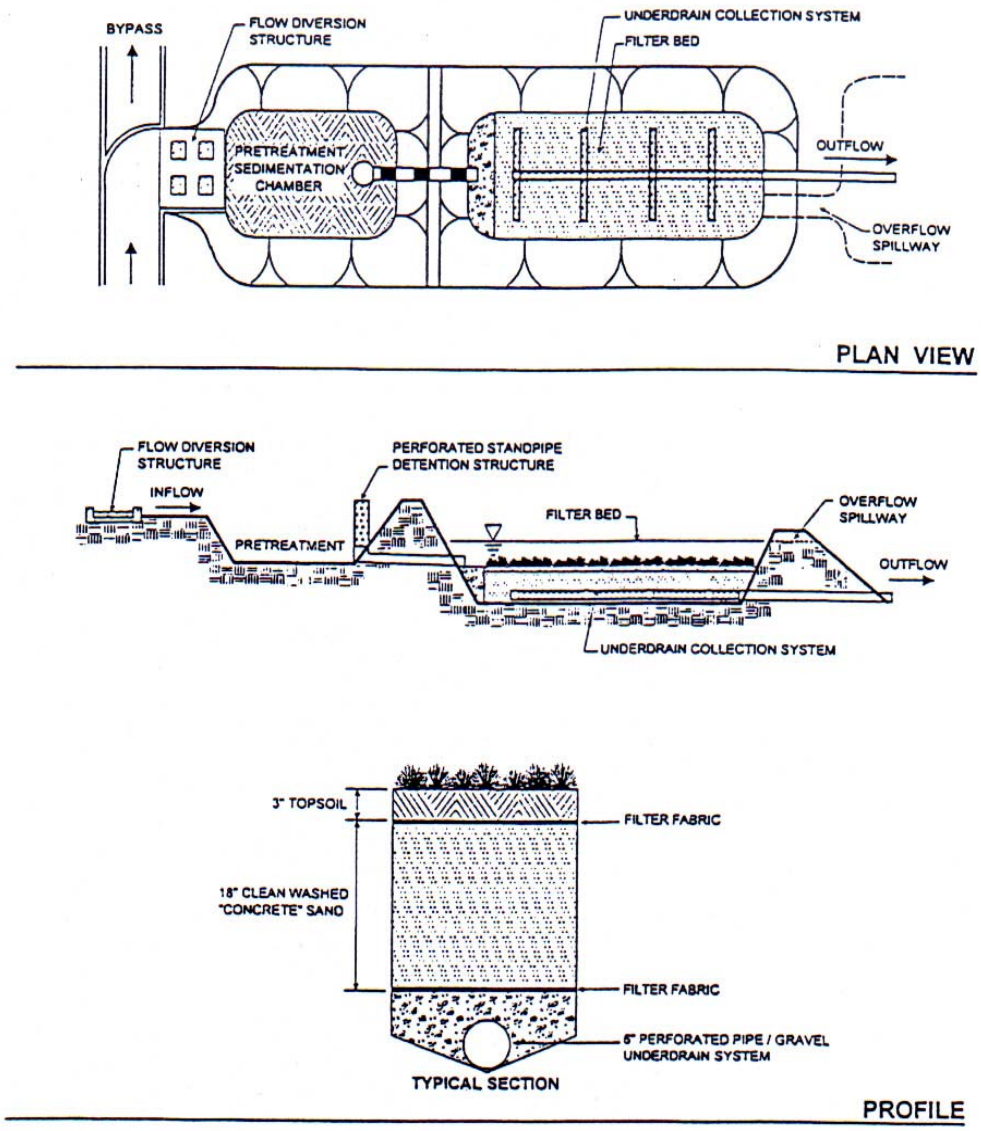


Figure 4.7
Surface Sand Filter

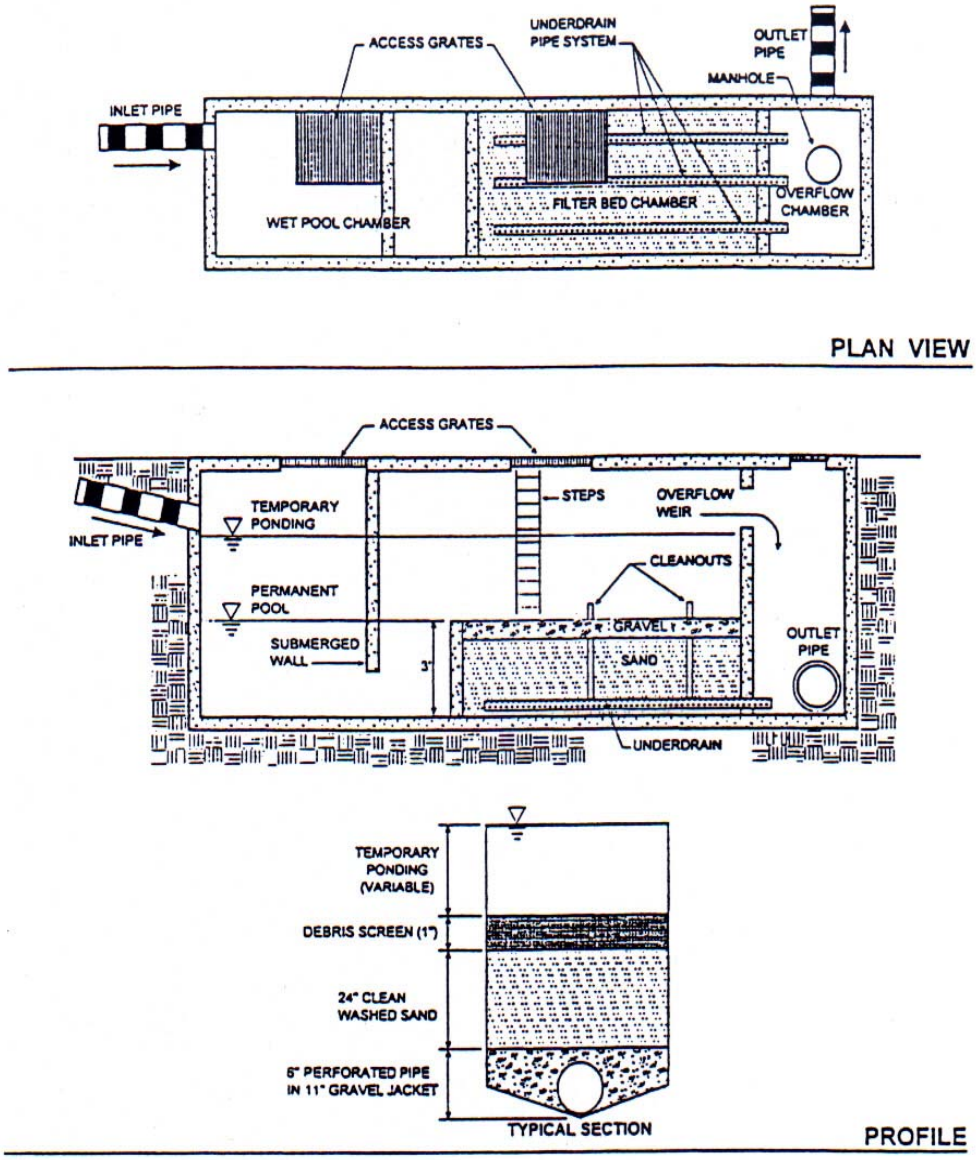
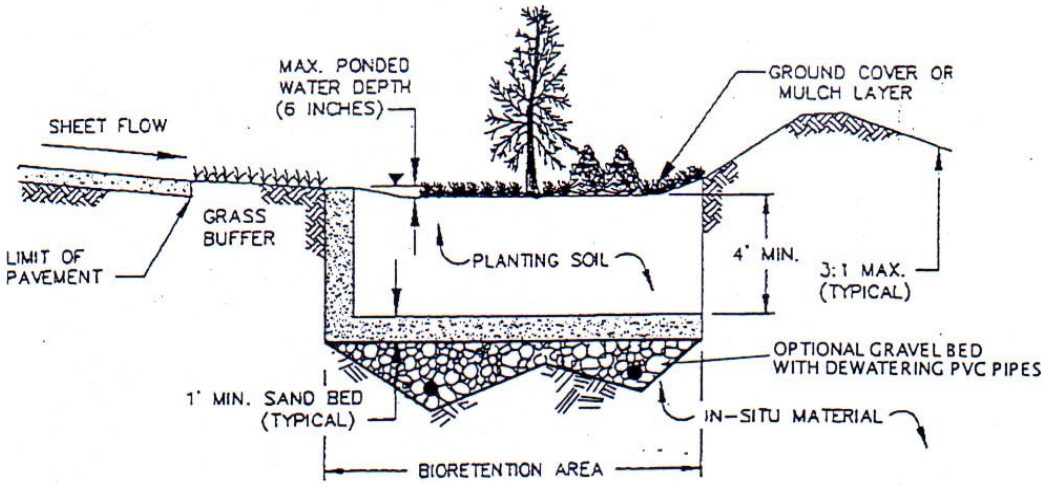


Figure 4.8
Underground Sand Filter



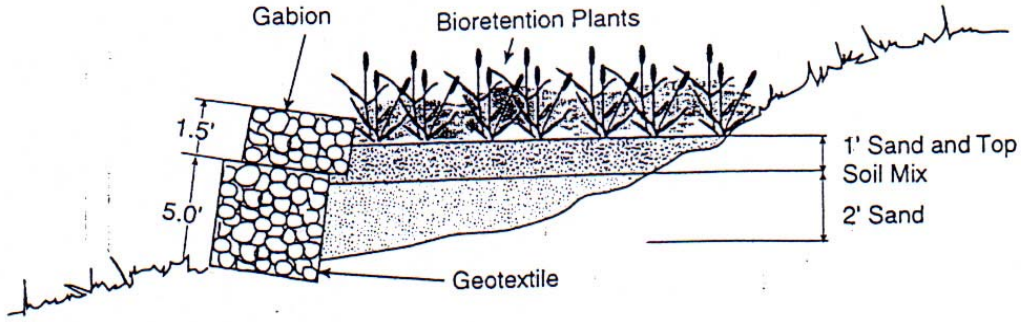


Figure 4.9
Bio-retention Facility

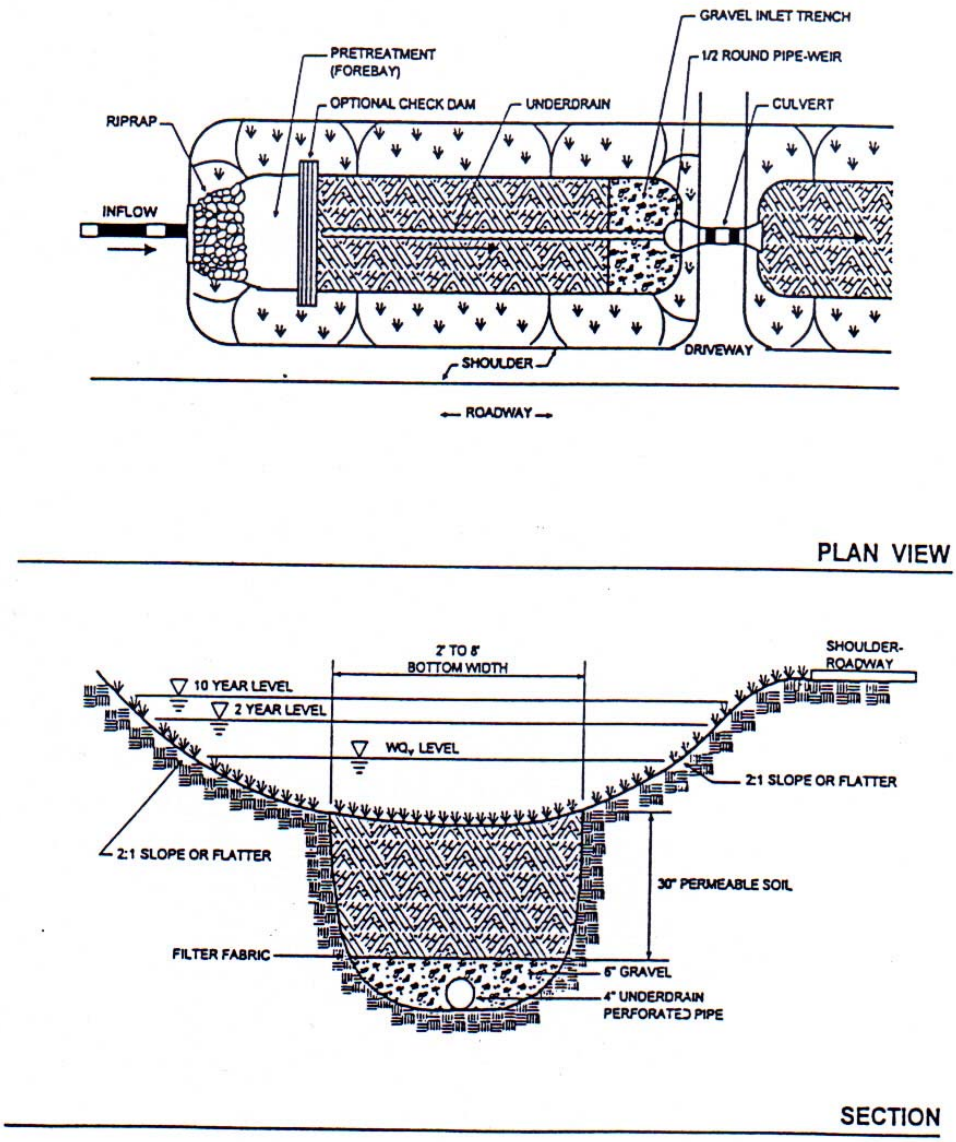


Figure 4.10
Water Quality Swale

Chapter 5
ECONOMIC ANALYSIS

The objectives of the economic analysis were to: 1) assess the implementation cost associated with the proposed standards; and 2) to qualitatively assess potential savings that might be realized through the use of low impact development. **NOTE: the recharge computations and assumptions used herein are based on the original developed plan standard of 1.5" for all impervious areas over 50% of the total site area. This criteria was later changed to 0.5" from all impervious areas, which is not reflected in the following analyses.**

Implementation Cost Assessment

To meet the first objective, ten (10) previously approved land development project sites within the Spring Creek Basin were reanalyzed using the standards proposed in Chapters 3 and 4.

Table 5.1 provides a general description of the ten (10) test sites used in the analysis. Five (5) of the ten (10) sites were provided by municipal engineers, and one was provided by The Pennsylvania State University. The four (4) remaining sites were selected to provide a variety of site land uses and to provide sites from multiple municipalities. Three (3) of the five (5) sites that were provided by municipalities are experiencing nuisance flooding or have questionable pre-development peak runoff rate estimates.

The test sites ranged in size from small commercial developments less than one (1) acre in size, to a site that included a drainage area of approximately 230 acres. The site land uses included commercial, institutional, light industrial, and single family and high-density residential developments and were located within five (5) different municipalities. In addition, two (2) of the ten (10) sites were Water Quality Sensitive (WQS) developments as defined in Chapter 3. The WQS developments included a gas station and a bus storage facility. Seven (7) of the sites originally had dry surface stormwater management ponds, one (1) site had a subsurface detention facility, one (1) site had a water quality retention pond, and one (1) site did not originally have to provide detention for peak runoff rate control. This well mixed group of developments had impervious areas ranging from 16 to 67 percent.

Table 5.1

Plans Analyzed Using the Proposed Technical Standards

Site No.	Pre-Development Land Use	Post-Development Land Use	Year	Site Area (acres)	Municipality	Known Problems	Original Computation Method	Original Return Periods Analyzed	Site Percent Impervious	Water Quality Sensitive Development
1	Agricultural	Commercial Small	94	1.1	College Twp	None	Modified Rational	1,10,100	59	NO
2	Agricultural	Commercial Small	99	3.02	College Twp	None	Modified Rational	1,10,100	64	NO
3	Institutional	Institutional	91	230	College Twp	Nuisance Flooding	TR-55	2,10,25,100	16	NO
4	Barren Land	Commercial Large	94	87	College Twp	Unsightly, Size	Modified TR-55	1,10,100	67	NO
5	Horse Farm	Residential High Density	95	16.4	Patton Twp	None	Modified Rational	1,10,100	50	NO
6	Agricultural	Residential Single Family	96	34.38	College Twp	Nuisance Flooding	Modified Rational	1,10,100	30	NO
7	Agricultural	Light Industrial	88	8.1	Ferguson Twp	None	TR-55 approx. Method	2,10,25,100	47	YES
8	Wooded	Residential Mobile Home	98	23.3	Patton Twp	None	TR-55	1,10,100	<30	NO
9	Urban	Gas Station	96	5.83	Spring Twp	None	Rational Method	1,10,100	64	YES
10	Open Space	Parking Lot	97	0.96	Benner Twp	None	Modified Rational	1,10,100	38	NO

Analysis

The first step in the analysis procedure was to verify the original computational analysis performed for each of the sites. This was done by actually duplicating the calculations using the same data, assumptions, model parameters, and model as the original designer. None of the reports had errors in the computations, and each was done in a somewhat similar manner consistent with existing ordinances. The computational methodologies were done by one of four analytical methods, each based on either the NRCS's TR-55 method or the rational method.

Following the initial verification, the sites were reanalyzed using the proposed standards and computational methodologies. The resulting post-development runoff hydrographs were then routed through the stormwater ponds to determine if the ponds were adequate in size, needed to be enlarged, or could be reduced in size. This analysis also verified if previously undetained areas could remain as such. If the ponds were found to be too small or large, the principle spillways were redesigned to determine a new optimum pond depth. If the ponds could not be made to work by adjusting the principle spillways alone, because they were now too small; additional depth (height) was added to the berm to increase the stage/storage capacity of the pond. None of the sites were redesigned to the extent that ponds or infrastructure were moved for the analysis.

Table 5.2 shows a comparison of the original report as pre-development peak runoff rates to the pre-development peak runoff rates computed by the proposed standards. The table also shows the percent difference between the original computed peaks for each site and the peaks computed using the proposed procedures. The proposed procedures specify a single consistent computational methodology. The general inconsistency in the values reported in Table 5.2 identifies that a significant variation in computational results occurs under existing ordinance provisions that is not attributable to site conditions. In addition, all but one pond overestimated the 1-year return period peak runoff rates. This over-estimation of the pre-development peak rates often results in downstream nuisance flooding, which is not an infrequent complaint of residents following the construction of stormwater management ponds.

Table 5.2

Originally Computed Pre-Development Peak Runoff Rates, Compared to Peak Runoff Rates Under the Proposed Standards

Site No.	Post- Development Land Use	Original Standards					New Standards					Difference (%) Between Original and Proposed Computed Pre-Development Runoff Rates for Return Periods (years)				
		Pre-Development Peak Runoff Rates (cfs) for Return Period (years)					Pre-Development Peak Runoff Rates (cfs) for Return Periods (year)									
		1	2	10	25	100	1	2	10	25	100	1	2	10	25	100
1	Commercial Small	0.5	na	1.7	na	3.4	0.2	0.4	1.5	3.5	4.9	-60	na	-12	na	44
2	Commercial Small	0.9	na	3.6	na	7.4	0.4	0.8	3.0	5.8	10.9	-58	na	-16	na	47
3	Institutional	na	139.2	355.4	464.2	660.2	6.9	9.5	38.6	82.8	156.2	na	-93	-89	-82	-76
4	Commercial Large	0.1	na	1.9	na	9.2	1.1	2.5	10.8	24.6	47.6	1000	na	468	na	417
5	Residential High Density	1.4	na	3.2	na	12.8	1.0	2.1	8.9	20.7	40.2	-28	na	180	na	213
6	Residential Single Family	3.3	na	13.4	na	31.8	1.9	3.6	12.3	25.5	49.8	-41	na	-8	na	57
7	Light Industrial	2.4	na	5.8	7.2	10.1	0.1	0.3	1.4	3.4	6.7	-94	na	-75	-53	-34
8	Residential Mobile Home	11.8	na	51.7	na	121.0	1.2	2.8	12.4	30.2	62.7	-90	na	-76	na	-48
9	Gas Station	1.8	na	2.9	na	4.4	1.0	1.3	2.2	3.0	4.3	-43	na	-24	na	-1
10	Parking Lot	1.5	na	3.4	na	6.7	0.8	1.6	4.7	8.0	13.6	-45	na	38	na	103

- Percent Difference
- Positive Value Indicates Proposed Standards Peak Runoff Rate Higher
- Negative Value Indicates Proposed Standards Peak Runoff Rate Lower

Of a more serious nature is the under-estimation of 100-year return period runoff rates under existing ordinances. If a true 100-year runoff event was to occur in the area, ponds with significantly undersized emergency spillways would be susceptible to failure. The statements regarding the accuracy of the original computed runoff estimates are made based on a knowledge of realistic historical flows from similar sized research watersheds outside the Spring Creek basin for which historical runoff data is available.

Following the analysis of the sites based on peak runoff rate, the sites were analyzed for the requirements of water quality and recharge. The stream bank morphology and storm drain design event criteria were not checked because they are considered here to be simple to implement without significant site modifications. For sites having stormwater facilities that had storage volumes greater than that required for peak runoff rate control, the required water quality volume was provided within the ponds. This was done to provide uniformity in the cost evaluations. The first (lowest) orifice invert of the principle spillway was set equal to the elevation equivalent to the water quality volume in the pond. The design events were then re-routed through the ponds, as required by the ordinance, to determine if the ponds still had adequate capacity. Again, the principle spillways were first altered, followed by an increase in the stage/storage depth (if required), to verify the pond discharges could still meet the peak rate criteria. The required additional recharge volumes were then computed, if required. However, it was assumed that these volumes would not affect the pond routings or designs.

As proposed, a capture volume is required for all stormwater management facilities. Because it was assumed that the water quality volume was accounted for in the ponds, the required capture volume was not evaluated because the capture volume was less than the water quality volume in all cases. Ponds that have water quality volumes fully accounted for in the pond designs meet the capture volume criteria by default.

Analysis Results

The results of the analysis can be found in Table 5.3. Table 5.3 indicates whether the pond size was adequate under the proposed standards and the ease of implementing the new peak runoff rate, water quality, and recharge criteria. Ease in this regard is related to how easy it

would be for a designer to implement the criteria into the design without changing the existing impervious area or basic design concept. Ease here is not a measure of cost. Cost will be addressed in the following section. It can be seen, from Table 5.3, that two of the ponds were considered too small. One of these ponds consisted of the small underground storage facility. It was found that the volume needed to be increased by approximately 20% to meet the proposed design criteria. This is because use of the modified rational method, that was used originally to design the pond, generally produces results that require less storage volume than the NRCS methods by approximately this same percentage. The second pond that was too small was due to poor assumptions made by the original designer that were, however, still within acceptable limits of the computational methods and the original ordinances on which the design was based.

The small development that originally did not need to provide stormwater management, still did not need to, under the proposed provisions. However, because the site was a WQS development, it must meet the specified water quality requirements. In general, only the site that used underground storage would have had difficulty enacting the water quality criteria, and that difficulty could be overcome with the use of specialized storm drain inlets. This same site would also have had difficulty with the recharge criteria because the impervious area following development was 64%. Designing a recharge system for the large commercial development that had 67% impervious area also would have been difficult. However, these ordinance criteria could be met at both sites.

Cost Comparisons

Costs associated with meeting the proposed standards for peak control, water quality, and recharge were analyzed. The actual pond cost was used for comparison in cases where this value was available. In cases where actual costs were not available, estimated costs were developed using data from recently bid projects. The costs for each of the three requirements (peak runoff rate, water quality, and recharge) are shown in Table 5.4. The large pond located on the institutional site (site number 3) did not require any additional costs, partially because it was originally constructed as a water quality pond. However, this does not indicate that the proposed standards would not have required some changes in the outlet control structure. The

Table 5.3

Ease of Implementing Proposed Standards into Original Site Design for Ten Test Sites

Site No	Post-Development Land Use	Existing Pond Type	Pond Size Evaluation Proposed Standards	Proposed Pond Design Return Period Required	Safely Passes 100-year Event	Ease of Implementing WQ Criteria	Ease of Implementing Recharge Criteria
1	Commercial Small	Dry Surface	OK*	25	NO*	Easy, In Pond	Moderate
2	Commercial Small	Subsurface	Too Small	25	NO	Difficult	Difficult
3	Institutional	WQ Wet	OK	100**	YES	Easy, In Pond	None
4	Commercial Large	Dry Surface	OK	50	YES	Easy, In Pond	Difficult
5	Residential High Density	Dry Surface	OK	25	YES	Easy, In Pond	None
6	Residential Single Family	Dry Surface	Too Small	25	NO	Easy, In Pond	None
7	Light Industrial	Dry Surface	OK*	25	NO*	Easy, In Pond	None
8	Residential Mobile Home	Dry Surface	OK	50	YES	Easy	None
9	Gas Station	None	Not Needed	Not Needed	Not Needed	Moderate	Moderate
10	Parking Lot	Dry Surface	OK	25	YES	Easy, In Pond	None

Site No	Notes Regarding Ease of Implementation
1	Would require minor pond modifications for safety
2	Would require design changes
3	Original pre-development outflows high, pond has adequate additional storage to correct with only spillway modification
4	Ordinance post-development outflows low, pond was originally over designed to correct
5	Would require limited design changes
6	Original pre-development outflows high, would require extensive design changes
7	WQSD, would require minor pond modifications for safety, site flat, may be difficult to implement WQ volume in pond
8	Original pre- and post-development outflows high, pond has adequate additional storage to correct
9	WQSD, would require use of WQ swales or other BMPs
10	Can fully meet new ordinance by simply altering principle spillway

* Would need additional berm height, 0.5 ft for freeboard for 100-year event

** Pond requirements from Chapter 105 permit

pre-development peak runoff rates computed under the proposed standards were significantly lower than those originally computed (this pond has been reported to be causing nuisance flooding). This difference could have been accounted for without any modification to the pond, except for the design of the principle spillway and the permanent pool water level. In order to meet the computational requirements required by the proposed standards, four (4) ponds needed to be redesigned (site numbers 1, 2, 6 and 7). In two of these cases (site numbers 1 and 7), the required modifications were simply a matter of increasing the berms 0.5 feet for freeboard requirements. Costs associated with the small site that had an underground detention facility (site number 2) would have increased 22% as a result of the need for another pipe section. The underground storage design criteria could have been waived by special exception by the Municipal Engineer (refer to computational methods section).

The most significant cost increase due to peak runoff rate comparisons was for one of the residential developments (site number 6). It was found that significant design changes would be required for peak runoff control, which resulted in a cost increase of 67% over the original construction costs. This same site was one of those provided by the municipality due to numerous complaints of nuisance flooding at the site. The nuisance flooding indicates that the site, as designed, does not adequately control peak runoff magnitudes.

Meeting the proposed water quality criteria resulted in an average increase in cost of 14% for the 10 sites (standard deviation of 11, range 0 to 36). The highest percent increases were for the WQS developments (site numbers 7 and 9), as would be expected. As indicated previously for this analysis, water quality features were added as an initial stage within the existing ponds. However, developers and designers could choose to implement the requirements with other types of BMPs. Each designer would need to determine if this cost could be increased or decreased depending on site-specific conditions.

Under the proposed recharge criteria, four (4) sites (site numbers 1, 2, 4, and 9) needed to be designed to account for recharge as a result of site impervious area. Site numbers 1 and 2 were assumed to have no increase in cost because a good designer could incorporate recharge easily into the design. Under the proposed standards, WQS development sites are not permitted to use structural recharge features. Therefore, a recharge feature was not

Table 5.4

Approximate Cost to Implement The Proposed Standards

Site No	Post-Development Land Use	Actual or Estimated Original Pond/BMP Cost	Estimated Additional "Proposed Standards"	Total Cost Increase "Proposed Standards (%)	Estimated Additional Cost for Peak Control	Cost Increase Peak Control (%)	Estimated Additional Cost for Water Quality	Cost Increase Water Quality (%)	Estimated Additional Cost for Recharge Criteria	Cost Increase Due to Recharge (%)
1	Commercial Small	$10,000	$1,000	10	$500	5	$500	5	$0	0
2	Commercial Small	$46,000	$13,000	29	$10,000	22	$3,500	8	$0	0
3	Institutional	$406,865	$0	0	$0	0	$0	0	$0	0
4	Commercial Large	$300,000	$50,000	17	$0	0	$30,000	10	$20,000	7
5	Residential High Density	$50,000	$8,000	16	$0	0	$8,000	16	$0	0
6	Residential Single Family	$30,000	$28,000	93	$20,000	67	$8,000	27	$0	0
7	Light Industrial	$14,000	$6,600	47	$1,600	11	$5,000	36	$0	0
8	Residential Mobile Home	$60,000	$8,000	13	$0	0	$8,000	13	$0	0
9	Gas Station	$0	$3,850	>100	$0	0	$3,300	>100	$550	>100
10	Parking Lot	$12,000	$1,600	13	$0	0	$1,600	13	$0	0

Note: Cost is not directly linked to difficulty because difficulty relates to ability to implement.

implemented for site 9. The $550.00 indicated in Table 5.4 for site 9 is assumed for the construction of another type of water quality BMP. The increased cost of meeting recharge criteria at the large commercial site (site number 4) increased stormwater infrastructure costs seven percent.

If the required water quality volumes in the ponds were not accounted for, the capture volumes would add approximately 25 to 75 percent of the water quality cost to the pond, depending on the proposed design. The lower percentage would apply to ponds that do not require dewatering devices such as subsurface drain lines.

Although the results of the analysis varied for a number of reasons as outlined above, it is concluded that **implementation of the proposed standards will result in an average 15 to 20 percent increase in the cost of stormwater infrastructure unless developers willingly choose to limit the amount of imperviousness of a site and maintain natural drainage-way ways.** Having said this, it is recognized that actual costs could vary significantly (from no increase to a significant increase) depending on specific site conditions and the ability of the design professional (with the owner's cooperation) to develop innovative approaches to stormwater quantity and quality control.

Qualitative Assessment Of Low Impact Development

The secondary objective of the economic analysis was to qualitatively assess general costs associated with the use of low impact development techniques. It is acknowledged that the economics driving land development activities are complex and include a wide variety of issues, some tangible and some non-tangible. The technical standards proposed in Chapters 3 and 4 are designed to encourage low impact development. However, stormwater management standards alone cannot mandate or force the use of these development techniques. This statement is supported by local municipal officials who concur that stormwater management criteria alone will not induce developers to use low-impact methods. Low impact development can only be mandated through restrictive zoning and land development ordinance provisions that can control overall site impervious area and dictate the use of low impact planning techniques.

It is further acknowledged that implementation of water quality requirements and recharge standards as an addition to standard peak runoff control standards will inherently add additional costs to land development activities. These additional costs will either be in the form of added infrastructure (see the previous section) or the need for additional land area to implement best management practices and low impact development techniques.

As an example, consider a commercial property in the Watershed that, under current ordinances, could be developed to a 70% impervious level (70% of the site could be covered with buildings and pavement). Given the terrain in many commercial areas within the Watershed, the only green space would be in the cut and fill slopes, the stormwater management pond, and miscellaneous parking area islands. The use of open channels and maintaining undisturbed buffers areas or minor drainage-ways would be nearly impossible on these sites. For these cases, the proposed requirements for water quality and recharge would make it impossible to decrease development costs. Unfortunately, data provided from the Centre Regional Planning Agency indicates that the majority of the planned development in the Shiloh Road Basin is commercial development with either 60% or 70% allowable imperviousness. If zoning is changed to force lower impervious coverage, it is the opinion of many municipal officials that developers will move to other locations or purchase larger plots of land, thus potentially inducing suburban sprawl. In addition, using site features such as bio-retention, porous pavement, or other water quality BMPs will always cost developers more than traditional methods when the total site impervious areas are not reduced.

The Shiloh Road Drainage Basin was used to facilitate the comparison of low impact versus traditional development methods. A detailed hydrologic analysis of the Shiloh Road Drainage Basin, included in Appendix E, clearly demonstrates that the net affect of the proposed computational standards will be to reduce runoff rates within the basin when compared to current ordinance provisions. Table 5.5 provides a comparison of predicted runoff rates under existing and proposed ordinance standards for the Basin. This flow comparison demonstrates that a significant environmental benefit related to a reduction in post-development peak runoff rates will be achieved through adoption of the proposed standards. This reduction in peak discharge rates will occur regardless of whether low impact development techniques are implemented.

Table 5.5
Shiloh Road Watershed Runoff Comparison
(Flow Values in cfs)

TR-20 Model	Return Period (Years)						
	1	2	5	10	25	50	100
Shiloh Road Existing Conditions	23	55	108	199	374	516	704
Post-development – Existing SWM Standards	66	129	209	331	543	713	924
Post-development – Proposed SWM Standards	23	58	106	201	375	516	687

The original objective of this analysis was to compare costs associated with development of the Shiloh Road Watershed under current zoning and subdivision regulations (considered traditional) with a low impact development approach (implementation of BMPs). Due to the limitations indicated above, this analysis is hypothetical in nature and is intended only as a general discussion of potential infrastructure savings associated with low impact development.

Data provided from the Centre Regional Planning Agency indicates that the majority of the planned development in the Shiloh Road Watershed is commercial development with either 60% or 70% allowable imperviousness (refer to Table 5.6 and Figure 5.1). The economic comparison was conducted for two of the larger tracts in the Shiloh Road Basin. The tracts are identified as tracts “A” and “B” for simplicity (see Figure 5.1). Tract A is a 108-acre parcel (tax parcel # 12-005-48), that is zoned office/commercial with an allowable impervious coverage of 60%; and Tract B, a 157-acre parcel (Tax parcels # 19-2-9 and 19-2-11). Traditional development layouts are shown in Figure 5.2 and low impact development methods in Figure 5.3 for these two tracts. These two tracts are used to identify factors influencing the relative cost differences between traditional and low-impact development techniques. A base assumption in the comparison is that there is no loss of square footage of developable area.

Tract A – Commercial/Office Development

Both of the development scenarios shown in Figures 5.2 and 5.3 have a total impervious area of 60% of the site. Both sites would require similar peak runoff control, water quality, and recharge designs. The water quality costs for the traditional development would be higher than the low impact design because no credits could be used for the traditional development. Recharge costs would be similar for both sites and would not be easy to implement (and therefore would be expensive) in either case because the site sits at the junction of several drainage-ways. In addition, it is estimated that the traditional development site in Figure 5.2 would require approximately 4,000 feet of large diameter storm drain piping in excess of the site storm drains and roof leaders that would be required for both the traditional and cluster development. Overall costs would be higher for the traditional design. In addition, the lower impact development shown in Figure 5.3 would have improved water quality, would maintain the health of the area to a higher degree, and would be more visually appealing (higher curb appeal).

Tract B - Residential Development

Both of the development scenarios shown in Figures 5.2 and 5.3 have a total impervious area of 30% of the site. Both development scenarios have 268 residential lots. The average lot sizes for the traditional site in Figure 5.2 are 0.5 acres in size compared to 0.25 acres for the cluster development in Figure 5.3. In the cluster development the total roadway length is reduced by approximately 15%. An associated reduction in the length of required storm drainage system would also be realized. Neither the traditional nor cluster development would need to meet a recharge requirement, and the cost of stormwater management ponds would be nearly identical for both developments. However, implementation of water quality criteria would be more costly for the traditional development scenario due to the fact that the low impact development could take advantage of the large retained natural areas for this purpose. Overall, development using low-impact, cluster designs will result in lower development costs.

Conclusions

The purpose of the economic analysis was to assess the increased costs associated with implementing the stormwater management standards and criteria proposed herein. Actual development sites were evaluated to provide a realistic estimate of the additional stormwater components required and their costs. In addition, a relative comparison of cost issues related to traditional versus low-impact development was presented. The following general conclusions are drawn from the analysis:

- Implementation of the proposed standards will result in an average 15 to 20 percent increase in the cost of stormwater infrastructure unless developers willingly choose to limit the amount of imperviousness on a site and maintain existing natural drainage-ways; and

- Implementation of low impact development techniques will generally result in a reduction in infrastructure costs.

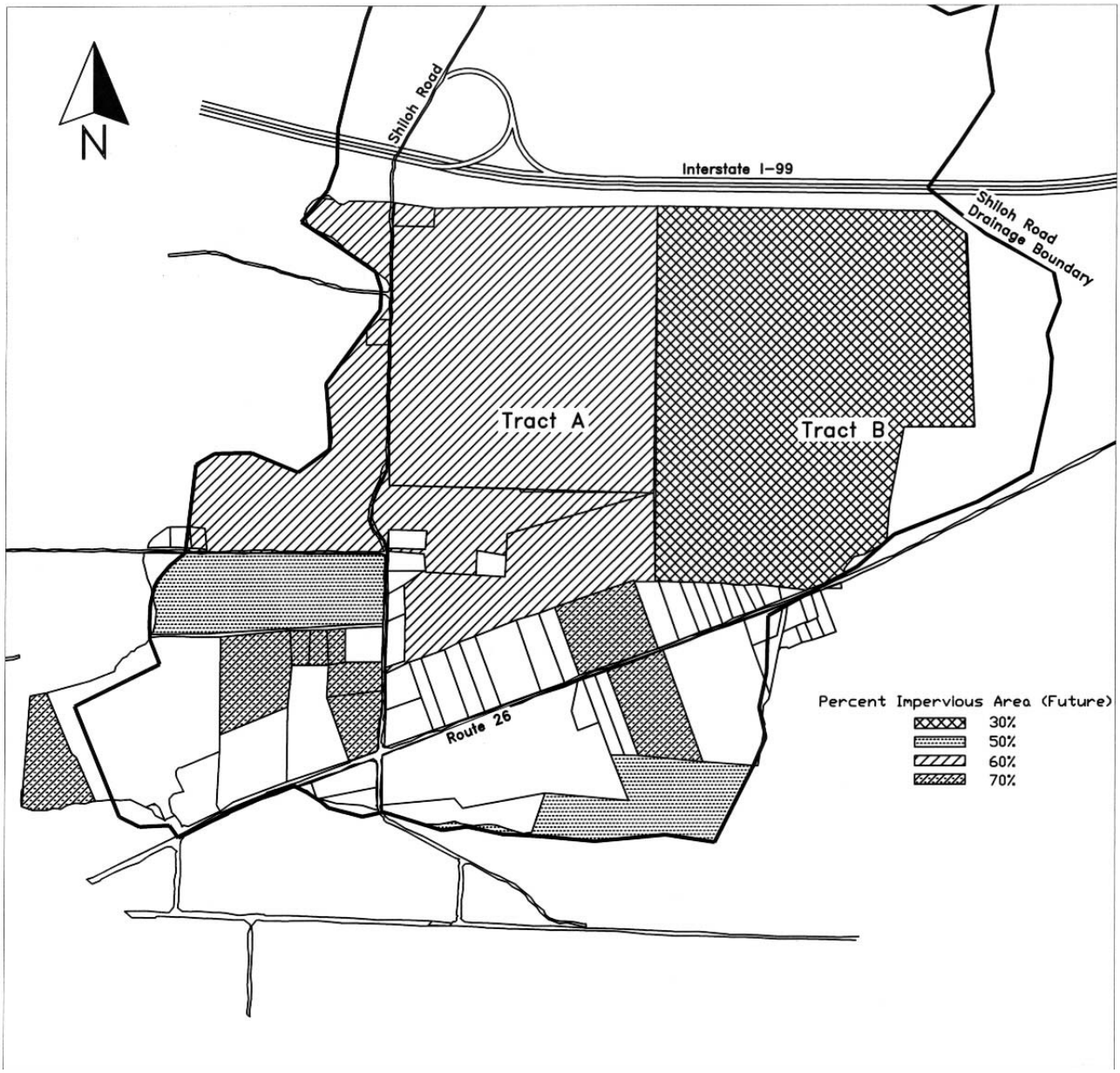


Figure 5.1

Percent Impervious Area (Future)

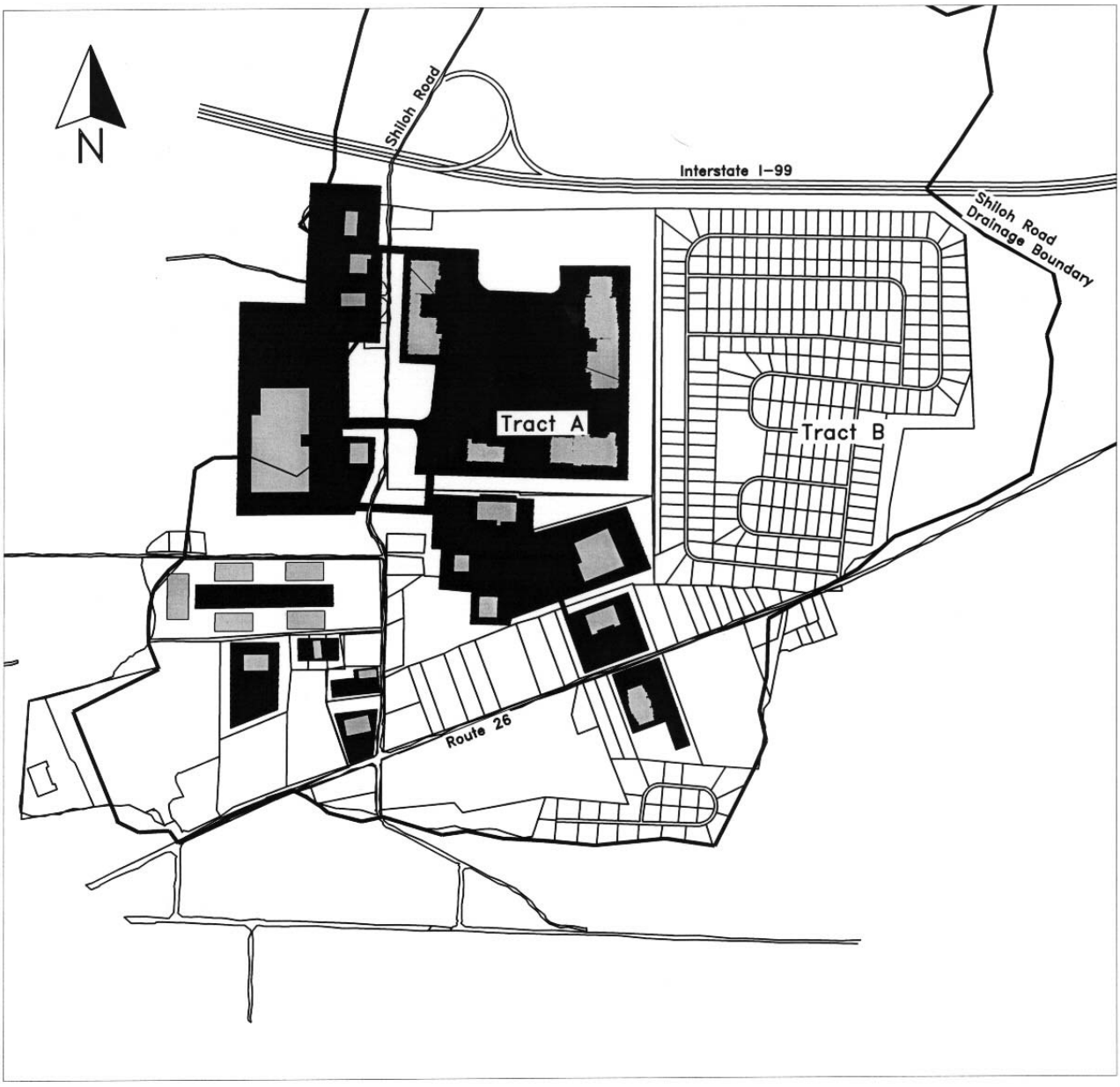


Figure 5.2

Traditional Development Layouts

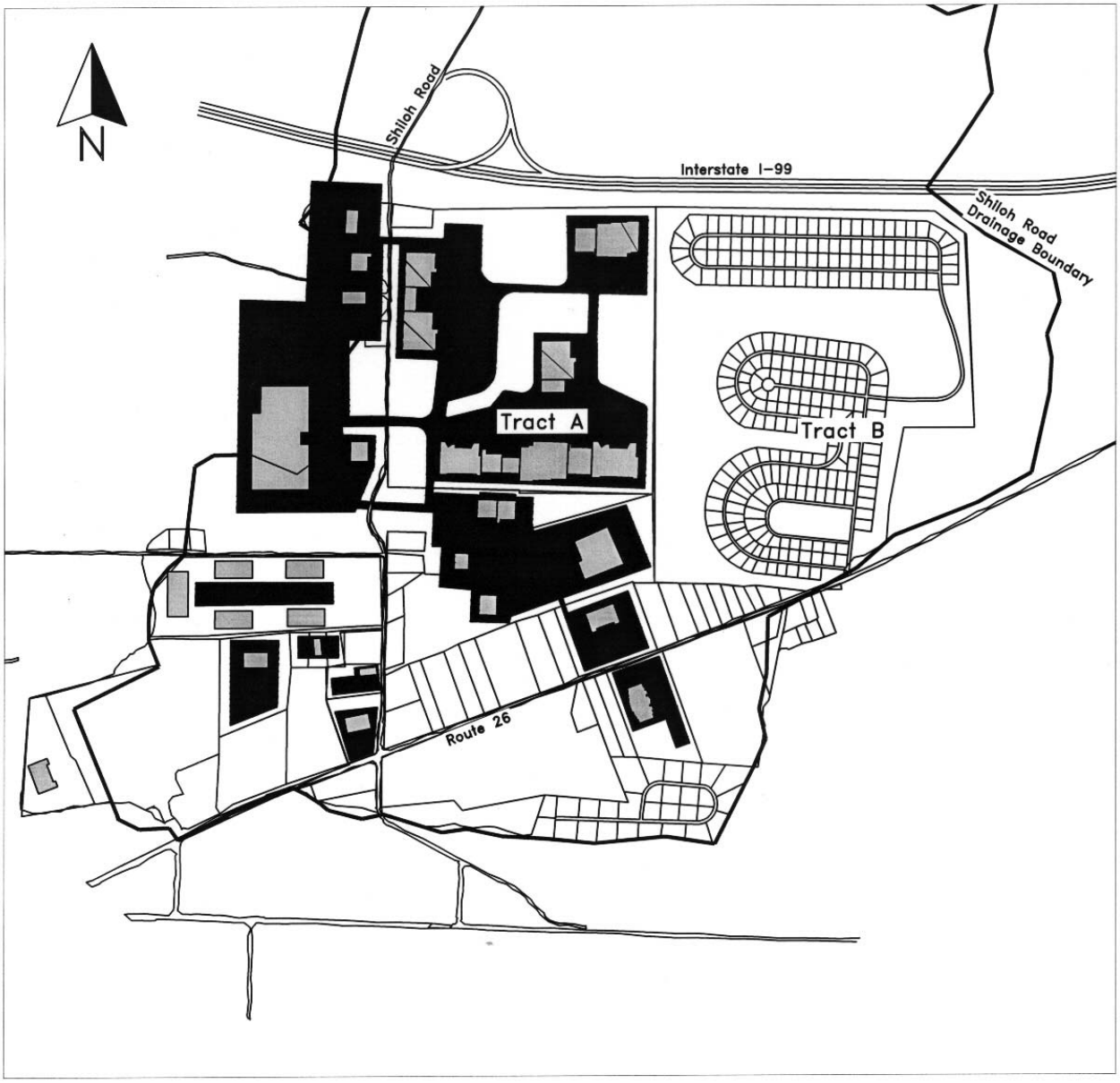


Figure 5.3

Low Impact Development Methods

Chapter 6

PLAN ADOPTION, IMPLEMENTATION, UPDATING, AND RECOMMENDATIONS

Introduction

This Stormwater Management Plan has been prepared in order to present the mechanisms through which the requirements of Pennsylvania Act 167 can be met throughout the Spring Creek Watershed. The specific standards and criteria presented in this Plan have been developed to represent a rationally derived set of requirements that must be satisfied by land developers in order to protect downstream persons and/or properties from damage from stormwater runoff originating at the development site. The measures to be taken by the developers include, but are not limited to, such actions as:

1. Assuring that the peak rate of stormwater runoff from the developed site does not exceed the peak rate occurring at the site prior to development; and
2. Managing the quantity, quality, velocity, and direction of resulting runoff in a manner that otherwise adequately protects health and property from possible injury.

The Plan also contains recommendations relative to the roles to be played by the various governmental agencies in the Watershed and presents specific ordinance provisions recommended for inclusion in municipal ordinances pertaining to the Watershed. These recommendations are presented in order to assist local governments in fulfilling their mandated role in the administration of stormwater management requirements within the Watershed. Since Act 167 places the ultimate responsibility for implementation and on-going administration and enforcement of stormwater management requirements in the hands of the local municipalities, each of the municipalities in the Watershed will be involved in plan implementation.

Plan Adoption and Implementation

The Spring Creek Watershed Stormwater Management Plan preparation process is complete with the Centre County Commissioners' adoption of the Plan and submission of the Plan to PA DEP for approval. Subsequent activities necessary to carry out the provisions of the Plan are considered by DEP to be part of the implementation of the Plan. The initial step in Plan implementation is DEP approval. Plan approval sets in motion the mandatory schedule of adoption of municipal ordinance provisions to implement the stormwater management standards and criteria. The municipalities within the Spring Creek Watershed will have six (6) months from the date of DEP approval in which to adopt the necessary ordinance provisions. Failure to do so could result in the withholding of all state funds to the municipality in accordance with Act 167.

Additional implementation activities are the development of a local program to coordinate DEP Chapter 105 and 106 Permit Application reviews and the development of a systematic approach for the correction of storm drainage problem areas.

County Adoption of the Plan

Formal adoption of the Plan is the responsibility of the Centre County Board of Commissioners. Under the requirements of Act 167, prior to the adoption of the Plan, the County will hold a public hearing pursuant to public notice of not less than two (2) weeks prior to the hearing date. Act 167 also stipulates that adoption of the Plan must be by a resolution carried by an affirmative vote of at least a majority of the members of the Centre County Commissioners. The resolution must refer expressly to the Plan and all associated maps, charts, and textural materials.

DEP Approval of the Plan

Upon adoption of the Plan by the Centre County Commissioners, the Plan is submitted to DEP for approval. The DEP review process involves the determination that all of the activities

specified in the approved Scope of Work have been satisfactorily completed in the Plan. Furthermore, DEP will only approve the Plan if it determines that:

1. The Plan is consistent with municipal floodplain management plans; state programs that regulate dams, encroachments, and other water obstructions; and state and federal flood control programs; and
2. The Plan is compatible with other watershed stormwater management plans for the basin in which the watershed is located and is consistent with the policies and purposes of Act 167.

DEP action to either approve or disapprove the Plan must take place within ninety (90) days of receipt of the Plan. Otherwise, the Plan would be approved by default. If the Plan is initially disapproved by DEP, it must be revised as necessary to secure approval and then resubmitted to DEP.

Municipal Adoption of Ordinance

The key ingredient for implementation of the Plan is the adoption of the necessary ordinance provisions by the municipalities within the Spring Creek Watershed. Alternately, Centre County could amend its Subdivision and Land Development Ordinance to include the proposed ordinance provisions. A Sample Stormwater Management Ordinance is provided as part of this Plan (see Appendix A). Municipalities can use the sample ordinance as a guide to modifying their existing ordinances, or the municipalities can adopt the sample ordinance “as is” with only minor modifications. The sample ordinance was developed following a review of existing stormwater management ordinances in the Spring Creek Basin and, therefore, incorporates most of the existing municipality ordinance features.

Because the Plan has been developed considering karst and non-karst areas, municipalities could apply the ordinance requirements to their entire municipality. This would even be true for areas of the municipality that are in another watershed as long as another Act 167 Plan was not in effect for those areas. In such a case, the more stringent of both Plan requirements would need to be used.

Local Program for Chapter 105 and 106 Permit Applications

Stream encroachments, stream enclosures, waterway diversions, and other activities regulated by Chapters 105 and 106 of DEP's Rules and Regulations may have an impact on the effectiveness of the runoff control strategy developed for the Spring Creek Watershed. Activities of these types may modify the conveyance characteristics of the Watershed and, therefore impact the relative timing of watershed peak flows and/or the ability of the conveyance facilities to safely transport peak flows. Therefore, to ensure that the DEP permitting process is consistent with the adopted and approved Watershed Plan, a local review of Chapter 105 and 106 applications should be coordinated with the DEP review process.

Development of a Systematic Approach for the Correction of Stormwater Drainage Problems

Correction of stormwater drainage problem areas in the Watershed is not specifically part of the Act 167 planning process. However, the development of this Stormwater Management Plan has provided the framework for the correction of problems because: 1) existing stormwater drainage problems within the Spring Creek Watershed have been identified; 2) implementation of the runoff control criteria specified in this Plan should prevent the existing drainage problems from becoming worse; and 3) the hydrologic modeling techniques used to formulate the runoff control criteria could be used as an analytical tool for identifying engineering solutions to major drainage problems.

In addition, this Plan can help correct stormwater drainage problems by opening the avenue of funding assistance through the PENNVEST program. The PENNVEST Act of 1988 provides low interest loans to governmental entities for the construction, storage, and infiltration of stormwater, and implementation of Best Management Practices to address point or non-point source pollution associated with stormwater. Municipalities within the Spring Creek Watershed should prioritize existing problems by severity, impact, and cost and consider the PENNVEST program for their financing.

Plan Updating Procedure

Under the requirements of Act 167, this Plan should be updated at intervals not exceeding five (5) years. More frequent updating of the Plan may be warranted if significant changes in the Watershed occur sooner. Factors that would warrant an update prior to the statutory date may include the following:

1. Changes to major stream or primary stormwater conveyance elements may occur that serve to affect Watershed hydraulics and possibly cause significant damage.

2. Peculiarities in the application of specific standards and criteria are experienced that interfere with the effective and equitable administration of the Plan requirements.

All of the involved agencies (Centre County Commissioners, the Municipalities, and DEP) should monitor conditions in the Watershed. In the event that any of the above listed conditions (or others) arise, the County should proceed with updating the Plan, as warranted.

Challenges to Successful Implementation and Enforcement of the Plan

Challenges to the successful implementation of this Stormwater Management Plan (PLAN) include obtaining political support for the Plan, ensuring uniform application and enforcement of the technical standards and regulations, and providing for consistent facility maintenance among the fourteen (14) jurisdictions within the Watershed.

The first obstacle will be overcoming the diverse political positions of each of the fourteen (14) municipalities with regard to development within their jurisdictions. Rural municipalities with limited commercial development may consider the PLAN overly rigid in its requirements. Others may feel that stormwater management facilities, especially surface ponds, are “eyesores” that cause more maintenance concerns then they alleviate, and “use up” valuable land. Some groups may also argue that the requirements of the PLAN place undue hardships on developers. Each of these can be a legitimate concern to those parties involved. Therefore, a

key element of the implementation program will be an educational outreach program for municipal officials to ensure that they understand how the proposed technical standards will impact drainage and water quality issues within their municipality.

Uniform application and enforcement of the proposed PLAN can only happen if all Municipal Engineers have a clear understanding of the hydrologic and hydrogeologic processes which are the basis for the technical standards and procedures in the PLAN. This Stormwater Management Plan has been developed considering, as much as practicable, the actual hydrologic processes that occur within the Watershed. In addition, the latest research findings and knowledge regarding stormwater management impacts, planning and design methods have been incorporated in the PLAN. This has resulted in the development of “non-traditional” computational requirements in the PLAN that will require educational programs to be developed for local planners and engineers. Municipal Engineers or their representatives will also need to be educated regarding the intent of the PLAN and how to reasonably implement the technical standards and criteria.

The third implementation issue identified above is providing consistent stormwater facility maintenance among the fourteen (14) municipalities within the Watershed. As with any infrastructure system (roads, sewers, waterlines, etc.) stormwater facility maintenance is a key component of the system's continued successful operation. This is particularly true of many water quality best management practices. In the past, local municipalities have required the construction of stormwater facilities as a part of development activities. However, with the exception of stormwater inlets and pipe systems constructed within dedicated rights of way, ownership and maintenance responsibility has been relinquished to the developers or homeowners groups. These individuals or groups are neither prepared for nor do they want the responsibility to maintain these facilities, and maintenance often falls to the municipality.

The need for uniform application of regulations and stormwater facility maintenance logically leads to the observation that a regional, watershed-wide, stormwater management authority or other regional entity be established. This issue is discussed further in the following section.

Consideration of a Regional Stormwater Management Authority

As identified in the previous section, one of the biggest problems with implementing a stormwater management plan of this magnitude is the large number of municipalities that will be involved. The 1996 International Countryside Stewardship Exchange sponsored by the Spring Creek Watershed Community noted, “because of the fragmented government, there is possibly a lack of a natural leader at a higher level that could serve to foster coordinated and cooperative efforts across the Watershed.” This fragmentation of municipal entities within the Watershed is the single largest deterrent to uniform implementation and enforcement of the proposed ordinance. Fragmentation will remain a problem in the future unless a cooperative effort is initiated among all municipalities within the Watershed to jointly enforce the Ordinance and take responsibility for maintaining the resulting stormwater infrastructure.

As a direct result of the 1996 International Countryside Stewardship Exchange, a joint municipal watershed commission was formed. The Spring Creek Watershed Commission consists of at least one (1) elected official from all municipalities within the Watershed, as well as the Centre County Commissioners. The purpose of this commission is to take a leadership role in advancing and coordinating projects and programs within the Spring Creek Watershed. It is recommended that this commission be the lead agency for facilitating cooperative efforts among all municipalities within the Watershed in an effort to provide uniform enforcement of regulations, and maintenance of stormwater infrastructure.

By implementing this PLAN, municipalities are in effect requiring developers to construct and maintain stormwater quantity and quality management facilities. Municipalities within the Watershed generally take over ownership of the roadway inlets and pipes or culverts within the right-of-way. However, they do not take over ownership and maintenance responsibility for other conveyance systems and other facilities such as stormwater and water quality BMP's. For residential developments, homeowner associations are generally burdened with the maintenance responsibility for these facilities. These associations are usually unaware of these maintenance responsibilities, and usually do not have the resources necessary to adequately maintain stormwater facilities.

Several large failures of stormwater management ponds, such as sinkhole collapses, have occurred in ponds within the Watershed that were under the maintenance authority of homeowners associations. In several of these instances it has taken over a year to conduct corrective measures. In the end, some municipalities are forced to take over the maintenance responsibilities of these stormwater facilities. Unfortunately, most of the municipalities are ill prepared (and already over burdened) to take over maintenance responsibilities, and even when a municipality assumes maintenance responsibilities, it still generally does not accept ownership of the facility.

Uniform and adequate maintenance of stormwater facilities is required to ensure adequate protection of the valuable ground and surface water resources within the Watershed. However, the issue of ownership and maintenance is complex, and unfortunately will remain so unless a regional stormwater management authority or other multi-municipal entity is created.

The creation of a regional stormwater authority is a viable option to alleviate maintenance concerns and unify the implementation and monitoring of the Plan. However, creating an authority would not be a simple task, and would require the cooperation of the municipalities involved in addition to the community. The justification for creating an authority and some of the issues, such as funding, that would need to be addressed are identified below:

- An authority could be staffed by personnel who have expertise in water quality issues and are knowledgeable regarding the hydrologic processes that occur within the Watershed.

- An authority could have a strong financial position, which would put them in a good position for acquiring State and Federal funding to initiate stormwater corrective actions.
- An authority could conduct monitoring and testing to determine major sources of pollution regarding stormwater facilities.

- An authority would have the ability to repair emergency conditions without financial concerns.

- An authority could have its own maintenance forces, which would be available to provide mowing and other routine maintenance on stormwater facilities.

- An authority would provide a single unified agency for enforcement of the Plan.

- An authority could develop and provide educational programs.

- An authority would be seen as representing the “watershed” and could have a larger and more effective level of involvement with local watershed or environmental organizations.

- An authority could work closely and cooperate with other authorities such as the County Water Authority.

- An authority would have the ability to purchase lands for regional stormwater management.

- The authority's area of responsibility could be the watershed, portions of the watershed, or possibly the entire County, if desired by the local municipalities.

- Municipalities would need to determine what level of involvement the authority played in the land development review process. Options are: providing maintenance only, providing technical expertise to the municipalities, or conducting the stormwater management plan reviews for the municipalities.
- If the authority were to conduct reviews, it would be in their best interest to have in-house qualified staff to maintain the level of expertise required for practical consistent application of the proposed standards and criteria.

- A Board of Directors consisting of representatives from each participating municipality would need to be created to oversee the stormwater management quantity and quality function.

- Issues such as how to initially take in existing facilities would have to be addressed.

- Ownership of the stormwater management facilities would need to be addressed.

- To support an authority, a monthly "Water Quality Fee" could be assessed to all residents and businesses. This fee could be a part of current sewer bills similar to current recycling fees.

- In addition, a "tap in" fee for development could be instituted for connecting into the storm network.

- If the authority were to conduct land development reviews, a review fee could be instituted.

Two options exist for the creation of such an authority; either creation of a new independent authority, or expansion of the responsibilities of an existing authority. The only practical existing authority that has the resources to take on this responsibility is the University Area Joint Authority (UAJA). UAJA already has staff and resources in place for maintenance of sewer conveyance systems and treatment facilities, which could be used for the maintenance of storm sewer facilities. They also have a vested interest in monitoring water quality within Spring Creek as it relates to permitting for treated sewer discharges. Expanding their responsibilities to include maintenance and operation of storm sewer facilities should, at least, be considered as an option.

Regardless of which option is selected, the creation of a joint municipal authority for the maintenance, and operation of stormwater quality and quantity control facilities would not only be beneficial for municipalities, but also would serve to improve the overall health of the Watershed through consistent and uniform application of regulations and facilities maintenance.

Recommendations

The goal of the Pennsylvania Stormwater Management Act of 1978 (Act 167) was to provide all municipalities within a specific watershed a technically sound and administratively consistent implementation procedure for comprehensive stormwater quantity and quality management.

The technical standards, computational procedures, and other requirements developed as a part of this plan meet this goal, and are designed to protect the surface and groundwater resources within the Spring Creek Watershed from stormwater quantity and quality impacts which result from development activities. Upon adoption by the Centre County Commissioners and approved by the Pennsylvania Department of Environmental Protection, all municipalities within the Watershed will be required to adopt these technical standards.

The following additional recommendations are made to ensure the success and continued effectiveness of this PLAN:

1. The Spring Creek Watershed Commission should continue its leadership role within the Watershed, and take the lead in the development of an appropriate watershed stormwater authority or other entity. This entity would be responsible for consistent and appropriate application and enforcement of these technical standards, and ownership and maintenance of all stormwater infrastructures within the Watershed.

2. To promote effective and responsible management of stormwater quality and quantity within the Spring Creek Watershed some subjectivity and flexibility has been built into the standards proposed in this plan. It is acknowledged that this flexibility may result in differences of opinion among design professionals with respect to the function, performance, and appropriateness of specific management practices. It is recommended that an arbitration panel be established to resolve these issues. This arbitration panel should be comprised of up to eight or more “experts” in the area of urban drainage design and stormwater management. When a dispute arises, the parties to the dispute would select a minimum of three (3) members of the arbitration panel to hear the case. The final decision of the arbitrators would be final.

   It is suggested that the Spring Creek Watershed Commission, which is comprised of elected officials from all municipalities within the watershed, be responsible for establishing policies and standards for the arbitration panel including selection of its members.

3. An education and training program needs to be developed and implemented to ensure that Municipal Engineers and reviewers as well as land development professionals doing work within the Watershed possess an appropriate level of competency in stormwater quantity and quality design. Design professionals and reviewers must have a thorough understanding of hydrologic and hydraulic processes occurring within the Spring Creek Watershed (due to its unique karst nature) prior to being permitted to certify stormwater plans.

4. The RBP III Assessment completed in Spring Creek during the summer of 1999 should be repeated at five (5) year intervals to monitor the general health of the Watershed.

5. Storm event water quality sampling is currently being conducted by the Spring Creek Watershed Community. When sufficient data becomes available, it is recommended that this data be used to calibrate EPA's Source Loading and Management Model, (SLAMM) or other appropriate model to assess the impact of BMPs on water quality within the Watershed. This analysis should be used to re-evaluate the BMP selection criteria provided in this study.

6. Stormwater impacts are directly related to the conversion of natural land areas to impervious surfaces during development activities. It is recommended that Municipal Zoning Ordinances and other regulations be modified to reduce required impervious surfaces wherever possible.

7. The Spring Creek Watershed contains many karst drainage-ways and enclosed depressions that serve important infiltration / recharge functions. These features are critically important to the interaction between surface and ground water within the Watershed. However, these areas often do not appear as channels or significant features, and have not historically been protected from development. These karst drainage features should be identified and protected by creating appropriate zoning and land development restrictions, or by placing them into permanent drainage easements. Enforcement of any such land use restriction should be implemented at the municipal level.

8. High quality agricultural land and wooded areas (particularly in upland areas of the Watershed) are often important groundwater recharge areas. Zoning ordinances should be modified to require hydrologic investigations of these areas prior to any rezoning action.

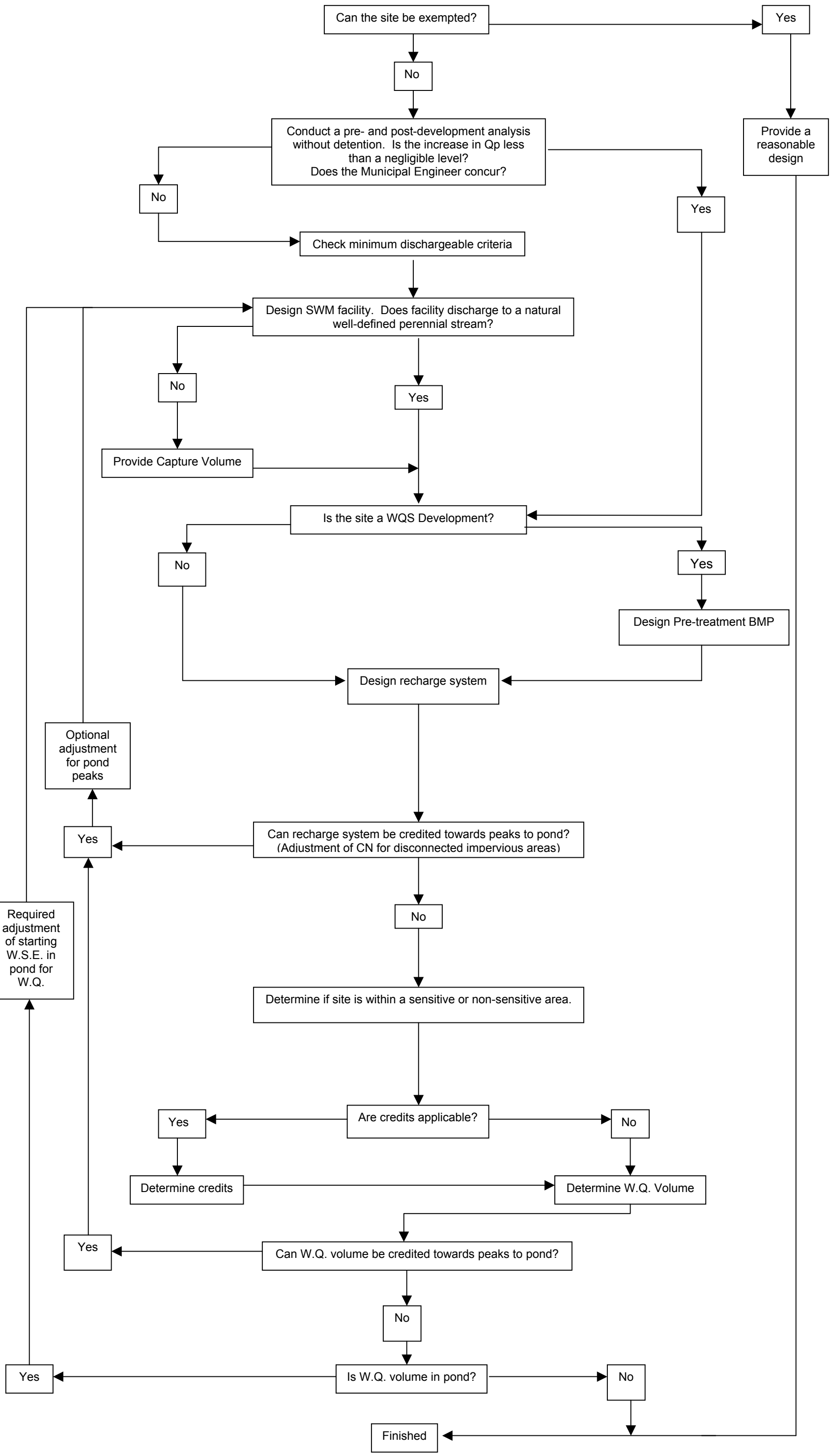


Exhibit 3.2
Stormwater Procedural Flowchart

ARTICLE I – GENERAL PROVISIONS

Section 101. Statement of Findings

The governing body of the Municipality finds that:

A. Inadequate management of accelerated stormwater runoff resulting from development throughout a watershed increases flood flows and velocities; contributes to erosion and sedimentation; overtaxes the carrying capacity of existing streams and storm sewers; greatly increases the cost of public facilities to convey and manage stormwater; undermines floodplain management and flood reduction efforts in upstream and downstream communities; reduces groundwater recharge; and threatens public health and safety.

B. This stormwater management ordinance has been developed considering the actual hydrologic processes that occur within the Spring Creek Watershed and how these processes can best be represented in hydrologic models that are used in engineering practice to model developing areas for stormwater management purposes. Process driven stormwater management planning is critical in the Spring Creek Watershed due to the underlying geology. The Spring Creek Watershed is mainly underlain by carbonate formations. As a result, significant areas within the watershed do not produce over-land or surface runoff, but rather contribute to the storm flow component of runoff in larger tributary areas via a greater than normal interflow component.

B. A comprehensive program of stormwater management, including reasonable regulation of development and activities causing accelerated erosion, is fundamental to the public health, safety, welfare, and the protection of the people of the Municipality and all the people of the Commonwealth, their resources, and the environment.

Section 102. Purpose

The purpose of this Ordinance is to promote health, safety, and welfare within the Spring Creek Watershed by minimizing the damages described in Section 101.A of this Ordinance through provisions designed to:

A. Manage accelerated runoff and erosion and sedimentation problems at their source by regulating activities that cause these problems.

B. Utilize and preserve the existing natural drainage systems.

C. Encourage the use of low impact development techniques to promote infiltration and groundwater recharge where appropriate.

D. Maintain existing flows and quality of streams and watercourses in the Municipality and the Commonwealth.

E. Preserve and restore the flood-carrying capacity of streams.

F. Provide proper maintenance of all permanent stormwater management facilities that are constructed in the basin.

G. Provide performance standards and design criteria for watershed-wide stormwater management and planning.

H. Protect groundwater and surface water quality.

Section 103. Statutory Authority

The Municipality is empowered to regulate land use activities that affect runoff by the authority by the Act of October 4, 1978 32 P.S., P.L. 864 (Act 167) Section 680.1 et seq., as amended, the "Storm Water Management Act", [and the applicable Municipal Code].

Section 104. Applicability

This Ordinance shall apply to those areas of the Municipality that are located within the Spring Creek Watershed, as delineated in Appendix B, which is hereby adopted as part of this Ordinance.

This Ordinance shall only apply to permanent stormwater management facilities constructed as part of any of the Regulated Activities listed in this Section. Stormwater management and erosion and sediment pollution control during construction activities are specifically not regulated by this Ordinance, but shall continue to be regulated under existing laws and ordinances.

This Ordinance contains only the stormwater management performance standards and design criteria that are necessary or desirable from a watershed-wide perspective. Local stormwater management design criteria (e.g. inlet spacing, inlet type, collection system design and details, outlet structure design, etc.) shall continue to be regulated by the applicable Municipal Ordinances or at the Municipal Engineer's discretion. However, these design criteria have also been provided within this sample Ordinance and may be adopted in full by municipalities.

The following activities are defined as "Regulated Activities" and shall be regulated by this Ordinance:

A. Land development;
B. Subdivision;
C. Construction of new or additional impervious or semi-pervious surfaces (roadways, driveways, parking lots, etc.);
D. Construction of new buildings or additions to existing buildings;
E. Diversion or piping of any natural or man-made stream channel;
F. Installation of stormwater management facilities or appurtenances thereto.

Section 105. Repealer

Any ordinance or ordinance provision of the Municipality inconsistent with any of the provisions of this Ordinance is hereby repealed to the extent of the inconsistency only.

Section 106. Severability

Should any section or provision of this Ordinance be declared invalid by a court of competent jurisdiction, such decision shall not affect the validity of any of the remaining provisions of this Ordinance.

Section 107. Compatibility with Other Ordinance Requirements

Approvals issued pursuant to this Ordinance do not relieve the Applicant of the responsibility to comply with or to secure required permits or approvals for activities regulated by any other applicable codes, rules, statutes, or ordinances.

Section 108. Landowner Responsibility

The granting of any exemption, permit, or approval by the Municipality does not relieve the applicant from assuring that stormwater runoff from the development site will not cause injury or damage to other persons or property.]

Section 109. References

Specific methods and publications indicated in this Ordinance shall, in all cases, refer to the latest available edition and include revisions or amendments thereto.

Section 110. Exemptions

Activities identified below are exempt from the requirement to submit a Stormwater Management Plan to the governing municipality for review. Exemption shall not relieve the applicant from implementing such measures as are necessary to protect health, safety, and property. These measures include adequate and safe conveyance of stormwater on the site and as it leaves the site. This exemption shall not relieve the applicant from meeting the special requirements for water quality and groundwater recharge for high quality (HQ) and exceptional value (EV) watersheds (DEP Chapter 93 and anti-degradation requirement), and Sections 304 C and E of this ordinance relative recharge and water quality volume requirements.

A. All development activities having impervious coverage of less than 10% of the total site area up to a maximum impervious area of 20,000 square feet. However, adequate and safe conveyance of stormwater from the site must be provided. For developments that are to be constructed in phases, the sum of all final phases must be considered in establishing exemption eligibility. Impervious cover shall include, but not be limited to, any roof, parking or driveway areas, and any new streets and sidewalks, or bikeways.

B. Land disturbance associated with the construction or alteration of one- and two-family dwellings, provided that the disturbance does not alter any stormwater condition beyond the boundaries of the lot or alter provisions of a previously approved Stormwater Management Plan for the lot or encompassing subdivision. Multiple (>2) lot subdivisions cannot be exempted.

C. Any site less than one (1) acre in size that decreases the total site impervious area following development, and:

- Is not located within a recognized sensitive area (as defined in Article II, Definitions, of this Ordinance);
- Is not defined as a water quality sensitive (WQS) development (as defined in Article II, Definitions, and Appendix B, Maps); or
- Is not located in an area where existing downstream stormwater problems are known to occur (the Municipal Engineer shall make the final determination as to pre-existing problems, but the municipality must have supporting documentation of past problems).

D. In addition, the Municipal Engineer may waive the requirement to prepare a stormwater management plan for sites larger than 1.0 acre for which the overall site impervious area is being decreased, and which meets the other conditions identified above.

The diversion or piping of any natural or man-made stream channel and/or for the installation of stormwater management facilities or modifications thereto cannot be exempted. These activities always require the submission of a Stormwater Management Plan. Exemptions A and B cannot be combined for use with small residential subdivisions.

In addition to the general exemptions identified above, exemptions for specific technical criteria are identified where applicable in Article III.

ARTICLE II – DEFINITIONS

For the purposes of this chapter, certain terms and words used herein shall be interpreted as presented below. Additional definitions are provided in the ACT 167 Plan definitions chapter.

A. Words used in the present tense include the future tense; the singular number includes the plural, and the plural number includes the singular; words of masculine gender include feminine gender; and words of feminine gender include masculine gender.

B. The word "includes" or "including" shall not limit the term to the specific example but is intended to extend its meaning to all other instances of like, kind and character.

C. The word "person" includes an individual, firm, association, organization, partnership, trust, company, corporation, or any other similar entity.

D. The words "shall" and "must" are mandatory; the words "may" and "should" are permissive.

E. The words "used or occupied" include the words "intended, designed, maintained, or arranged to be used, occupied or maintained".

Agricultural Activities – The work of producing crops and raising livestock including tillage, plowing, disking, harrowing, pasturing and installation of conservation measures. Construction of new buildings or impervious area is not considered an agricultural activity.

Alteration – As applied to land, a change in topography as a result of the moving of soil and rock from one location or position to another; also the changing of surface conditions by causing the surface to be more or less impervious; land disturbance.

Applicant – A landowner or developer who has filed an application for approval to engage in any Regulated Activities as defined in Section 104 of this Ordinance.

BMP (Best Management Practice) – Stormwater structures, facilities and techniques to maintain or improve the water quality of surface runoff.

Buffer Area – Area that is protected from development in order to prevent degradation of the water body or water quality.

Capture Depth – Depth of runoff captured from a given area and either allowed to evaporate, infiltrate, or be discharged through a spillway at a negligible rate.

Carbonate – A sediment formed by the organic or inorganic precipitation of mineral compounds characterized by the fundamental chemical ion CO_3, the principal element in limestone and dolomite strata.

Channel – A perceptible natural or artificial waterway, which periodically or continuously contains moving water having a definite bed and banks, which confine the water.

Closed Or Undrained Depression – In a Karst geologic area a distinct bowl-shaped depression in the land surface; size and amplitude are variable; drainage is internal. It differs from a sinkhole in that the ground surface is unbroken and usually occurs in greater density per unit area.

Conservation District – The Centre County Conservation District.

Credits – A deduction from the required amount. In this ordinance, implies reduction of required water quality volumes due to using a recommended practice.

Dam – An artificial barrier, together with its appurtenant works, constructed for the purpose of impounding or storing water or another fluid or semifluid, or a refuse bank, fill or structure for highway, railroad or other purposes which does or may impound water or another fluid or semifluid.

Design Storm – The magnitude and temporal distribution of precipitation from a storm event measured in probability of occurrence (e.g., a 5-year storm) and duration (e.g., 24 hours), used in the design and evaluation of stormwater management systems.

Designee – The agent of a Planning Commission and/or agent of the governing body involved with the administration, review or enforcement of any provisions of this ordinance by contract or memorandum of understanding.

Detention Basin – An impoundment structure designed to manage stormwater runoff by temporarily storing the runoff and releasing it at a predetermined rate.

Developer – A person, partnership, association, corporation, or other entity, or any responsible person therein or agent thereof, that undertakes any Regulated Activity of this Ordinance.

Development Site – The specific tract of land for which a Regulated Activity is proposed.

Dolomite – (1) A mineral consisting of calcium magnesium carbonate found as compact lime stone; or (2) limestone or marble rich in magnesium carbonate.

Downslope Property Line – That portion of the property line of the lot, tract, or parcels of land being developed located such that all overland or pipe flow from the site would be directed towards it.

Drainage Conveyance Facility – A Stormwater Management Facility designed to transmit stormwater runoff and shall include streams, channels, swales, pipes, conduits, culverts, storm sewers, etc.

Drainage Easement – A right granted by a landowner to a grantee, allowing the use of private land for stormwater management purposes.

Drainage Permit – A permit issued by the Township governing body after the drainage plan has been approved. Said permit is issued prior to or with the final Township approval.

Drainage Plan – The documentation of the stormwater management system, if any, to be used for a given development site, the contents of which are established in Section 403.

Drainage-way – The natural or man-made path of surface water from a given area.

Erosion – The movement of soil particles by the action of water, wind, ice, or other natural forces.

Erosion and Sediment Pollution Control Plan – A plan that is designed to minimize accelerated erosion and sedimentation.

Exfiltration – The process by which water or moisture moves from a subsurface trench, bed, or other feature into the subsoil. Exfiltration is best measured by a soil's percolation rate.

Existing Conditions – The initial condition of a project site prior to the proposed construction.

Flood – A general but temporary condition of partial or complete inundation of normally dry land areas from the overflow of streams, rivers, and other waters of this Commonwealth.

Floodplain - Any land area susceptible to inundation by water from any natural source or delineated by applicable Department of Housing and Urban Development, Federal Insurance Administration Flood Hazard Boundary - Mapped as being a special flood hazard area.

Floodway – The channel of the watercourse and those portions of the adjoining floodplains that are reasonably required to carry and discharge the 100-year frequency flood. Unless otherwise specified, the boundary of the floodway is as indicated on maps and flood insurance studies provided by FEMA. In an area where no FEMA maps or studies have defined the boundary of the 100-year frequency floodway, it is assumed - absent evidence to the contrary - that the floodway extends from the stream to 50 feet from the top of the bank of the stream.

Forest Management/Timber Operations – Planning and activities necessary for the management of forestland. These include timber inventory and preparation of forest management plans, silvicultural treatment, cutting budgets, logging road design and construction, timber harvesting, site preparation and reforestation.

Freeboard – A vertical distance between the elevation of the design high-water and the top of a dam, levee, tank, basin, or diversion ridge. The space is required as a safety margin in a pond or basin.

Grassed Waterway – A natural or constructed waterway, usually broad and shallow, covered with erosion-resistant grasses, used to conduct surface water from cropland.

Groundwater Recharge – Replenishment of existing natural underground water supplies.

Impervious Surface – A surface that prevents the percolation of water into the ground.

Impoundment – A retention or detention basin designed to retain stormwater runoff and release it at a controlled rate.

Infiltration Rate – The infiltration rate of a soil is related to the soil's final infiltration capacity and represents the rate at which water enters the soil/air interface at the top of the soil profile. Infiltration rates are measured in units of length / time.

Inlet – A surface connection to a closed drain. A structure at the diversion end of a conduit. The upstream end of any structure through which water may flow.

Interceptor – A channel, berm, or dike constructed across a slope for the purpose of intercepting stormwater, reducing the velocity of flow, and diverting it to outlets where it may be disposed.

Karst – A type of topography that is formed over limestone, dolomite, or gypsum by bedrock solution, and that is characterized by closed depressions or sinkholes, caves, and underground drainage (from AGI, Glossary of Geology, 1972).

Land Development – (i) The improvement of one lot or two or more contiguous lots, tracts, or parcels of land for any purpose involving (a) a group of two or more buildings, or (b) the division or allocation of land or space between or among two or more existing or prospective occupants by means of, or for the purpose of streets, common areas, leaseholds, condominiums, building groups, or other features; (ii) Any subdivision of land; (iii) Development in accordance with Section 503(1.1) of the Pennsylvania Municipalities Planning Code.

Land/Earth Disturbance – Any activity involving grading, tilling, digging, or filling of ground or stripping of vegetation or any other activity that causes an alteration to the natural condition of the land.

Land Use – The primary application employed in an area.

Limestone – A rock that, by accumulation of organic remains, consists mainly of calcium carbonate.

Lineaments – Straight or gently curved, lengthy features frequently expressed topographically as depressions or lines on the earth's surface. They can be more easily observed at a height of 100 meters or more and are usually found by researching aerial photographs or satellite photography. They are usually located in areas of faulting or in dense jointing along some rock stratigraphy.

Main Stem (Main Channel) – Any stream segment or other runoff conveyance facility used as a reach in the Spring Creek hydrologic model.

Minimum Allowable Discharge – In relation to this Stormwater Management Ordinance, the minimum rate that can be discharged for any drainage area for design storm events up to and including the 10-year event regardless of the modeled pre-development runoff estimate.

Municipality – Any of the several municipalities within the basin consisting of: Bellefonte Borough, Benner Township, Boggs Township, Centre Hall Borough, College Township, Ferguson Township, Halfmoon Township, Harris Township, Milesburg Borough, Patton Township, Potter Township, Spring Township, State College Borough, and Walker Township, Centre County, Pennsylvania.

Municipal Engineer – A professional engineer licensed in the Commonwealth of Pennsylvania and duly appointed by the subject municipality as their representative. In the event that a Stormwater Utility is formed, all references to the Municipal Engineer shall be considered to also imply the Stormwater Utility Engineer.

Natural Conservation Areas – A natural area protected during development for its water quality or recharge enhancing abilities.

Outfall – Point where water flows from a conduit, stream, or drain.

Outlet – Points of water disposal from a stream, river, lake, tidewater or artificial drain.

PA DEP – Pennsylvania State Department of Environmental Protection.

PA DOT – Pennsylvania State Department of Transportation.

Peak Discharge – The maximum rate of stormwater runoff from a specific storm event.

Percolation Rate – The rate at which water moves through a soil profile. Percolation rates are measured in units of time / length.

Pipe – A culvert, closed conduit, or similar structure (including appurtenances) that conveys stormwater.

Planning Commission – The planning commission of a municipality.

Point Discharge – The discharge from a pipe or channel that concentrates runoff at a single area.

Recharge Volume – The volume of water that is required to be recharged from developed sites.

Registered Professional – An individual registered in and licensed by the State of Pennsylvania including, for the purposes of this Ordinance, land surveyors, landscape architects, architects and engineers.

Regulated Activities – Actions or proposed actions that have an impact on stormwater runoff and that are specified in Section 104 of this Ordinance.

Retention Basin – An impoundment in which stormwater is stored and not released during the storm event. Stored water may be released from the basin at some time after the end of the storm.

Return Period – The average interval, in years, within which a storm event of a given magnitude can be expected to recur. For example, the 25-year return period rainfall has a 4% probability of occurring in any given year.

Runoff – Any part of precipitation that flows over the land surface.

Safe Passage – The routing of peak runoff events, usually the 100-year design event, safely through a structure without failure of that structure.

Scour – Generally refers to the change in a channel configuration provoked by sediment imbalance, due to natural or man made causes, between the supply and transport capacity of the channel.

Sediment Basin – A barrier, dam, retention or detention basin located and designed to retain rock, sand, gravel, silt, or other material transported by water.

Sensitive (Water Quality) Area – An area protected because development within that area could potentially cause contamination of groundwater reservoirs. These sensitive land areas are defined in Appendix B, Exhibit-1.

Sheet Flow – Runoff that flows over the ground surface as a thin, even layer, not concentrated in a channel.

Sinkhole – A localized, gradual or rapid sinking of the land surface to a variable depth, occurring in areas of carbonate bedrock; generally characterized by a roughly circular outline, a distant breaking of the ground surface and downward movement of soil into bedrock voids.

Spillway – A depression in the embankment of a pond or basin which is used to pass peak discharge greater than the maximum design storm controlled by the pond.

Stabilization – The proper placing, grading and/or covering of soil, rock or earth to ensure their resistance to erosion, sliding or other movement.

Storm Sewer – A system of pipes and/or open channels that convey intercepted runoff and stormwater from other sources, but excludes domestic sewage and industrial wastes.

Stormwater Management Facility – Any structure, natural or man-made, that, due to its condition, design, or construction, conveys, stores, or otherwise affects stormwater runoff. Typical stormwater management facilities include, but are not limited to, detention and retention basins, open channels, storm sewers, pipes, and infiltration structures.

Stormwater Management Plan – The plan for managing stormwater runoff in the Spring Creek Watershed adopted by the Centre County Commissioners as required by the Act of October 4, 1978, P.L. 864, (Act 167), and known as the "Spring Creek Watershed Action 167 Stormwater Management Plan.

Strata – Tabular or sheet-like mass, distinct layers of homogenous or gradational sedimentary material (consolidated rock or unconsolidated earth) of any thickness, visually separable from other layers above and below by a discrete change in the character of the material deposited or by a sharp physical break deposition or both.

Stratigraphic Unit – A stratum or body of strata recognized as a unit in the classification of the rocks of the earth's crust with respect to any specific rock character, property, attribute or for any purpose such as description, mapping, and correlation.

Structural Fill – For the purposes of this ordinance, shall imply any soil mass that is compacted in lifts to some tested criteria (standard or modified proctor) such as those under foundations or adjacent to retaining walls. Areas that for several years after construction respond to precipitation events similar to impervious areas.

Subarea – The smallest drainage unit of a watershed for which stormwater management criteria have been established in the Stormwater Management Plan.

Swale – A natural low-lying stretch of land or minor man made conveyance channel, which gathers or carries surface water runoff.

SWM – Stormwater management.

Topography – The general configuration of a land surface or any part of the earth's surface, including its relief and position of its natural and man-made features. The natural or physical surface features of a region, considered collectively as to its form.

Undetained Area – An area of a site that cannot be routed to a stormwater management facility because of its location. Generally small areas around access drives or below stormwater management facilities.

Water Quality Depth – Depth of precipitation required to be used in computing the water quality volume based on the percentage of imperviousness of a site.

Water Quality Sensitive (WQS) Development – Land development projects that have a high potential to cause catastrophic loss to local water quality and could potentially threaten ground water reservoirs. See Section 302 for additional definition.

Water Quality Volume – Volume of runoff required to be controlled from a site in a water quality BMP.

Watershed – The entire region or area drained by a river or other body of water, whether natural or artificial, a drainage basin or sub-basin.

Waters of the Commonwealth – Any and all rivers, streams, creeks, rivulets, ditches, watercourses, storm sewers, lakes, dammed water, wetlands, ponds, springs, and all other bodies or channels of conveyance of surface and underground water, or parts thereof, whether natural or artificial, within or on the boundaries of this Commonwealth.

Water Table – Upper surface of a layer of saturated material in the soil.

Wetland – Those areas that are inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions, including swamps, marshes, bogs, ferns, and similar areas.

ARTICLE III – STORMWATER MANAGEMENT

Section 301. General Requirements

A. All regulated activities in the Spring Creek Watershed which do not fall under the exemption criteria shown in Section 110 of this Ordinance shall submit a drainage plan to the governing municipality for review. This plan must be consistent with the Spring Creek Watershed Stormwater Management Plan. These criteria shall apply to the total proposed development even if development is to take place in phases. Impervious cover shall include, but not be limited to, any roof, parking or driveway areas, and any new streets and sidewalks. Any areas designed to initially be gravel or crushed stone shall be assumed to be impervious for the purposes of comparison to the waiver criteria.

B. Stormwater drainage systems shall be provided in order to permit unimpeded flow along natural watercourses, except as modified by stormwater management facilities or open channels consistent with this Ordinance.

1. Stormwater management facilities and related installations also shall be provided:

a. To ensure adequate drainage of all low points along the curb line of streets.

b. To intercept stormwater runoff along streets at intervals reasonably related to the extent and grade of the area drained, and to prevent substantial flow of water across intersections or flooded intersections during storms, in accordance with the procedures contained in Design Manual Part 2 (DM-2), Chapter 10, of the Pennsylvania Department of Transportation (PA DOT).

c. To ensure adequate and unimpeded flow of stormwater under driveways in, near, or across natural watercourses or drainage swales. Suitable pipes or other waterways shall be provided as necessary.

d. To properly drain stormwater runoff from all land development projects, except as required by recharge criteria. All lot and open areas shall be designed to drain to the nearest practical street or drainage system, existing or proposed, as defined by the respective Municipal Engineer, with no impact on adjoining properties, unless an area specifically designed for stormwater detention is provided.

C. The existing points of concentrated drainage that discharge onto adjacent property shall not be altered without permission of the altered property owner(s) and shall be subject to any applicable discharge criteria specified in this Ordinance.

D. Areas of existing diffused drainage discharge shall be subject to any applicable discharge criteria in the general direction of existing discharge, whether proposed to be concentrated or maintained as diffused drainage areas, except as otherwise provided by this Ordinance. If diffused flow is proposed to be concentrated and discharged onto adjacent property, the Developer must document that adequate downstream

conveyance facilities exist to safely transport the concentrated discharge, or otherwise prove that no erosion, sedimentation, flooding or other harm will result from the concentrated discharge.

E. Where a development site is traversed by watercourses, drainage easements shall be provided conforming to the line of such watercourses. The terms of the easement shall prohibit excavation, the placing of fill or structures, and any alterations that may adversely affect the flow of stormwater within any portion of the easement. Also, maintenance, including mowing of vegetation within the easement shall be required, except as approved by the appropriate governing authority.

F. When it can be shown that, due to topographic conditions, natural drainageways on the site cannot adequately provide for drainage, open channels may be constructed conforming substantially to the line and grade of such natural drainageways. Work within natural drainageways shall be subject to approval by PA DEP through the Joint Permit Application process, or, where deemed appropriate by PA DEP, through the General Permit process.

G. Any stormwater management facilities regulated by this Ordinance that would be located in or adjacent to waters of the Commonwealth or wetlands shall be subject to approval by PA DEP through the Joint Permit Application process, or, where deemed appropriate by PA DEP, the General Permit process. When there is a question whether wetlands may be involved, it is the responsibility of the Developer or his agent to show that the land in question cannot be classified as wetlands, otherwise approval to work in the area must be obtained from PA DEP.

H. Any stormwater management facilities regulated by this Ordinance that would be located on State highway rights-of-way shall be subject to approval by the Pennsylvania Department of Transportation (PA DOT).

I. Minimization of impervious surfaces and infiltration of runoff through seepage beds, recharge trenches, etc., are encouraged, where soil conditions permit, to reduce the size or eliminate the need for detention facilities.

J. To promote over-land flow and infiltration/percolation of stormwater, roof drains should not be connected to streets, sanitary or storm sewers, or roadside ditches unless approved by the municipal authority on a case-by-case basis.

K. Where deemed necessary by the respective Municipal Engineer, the applicant shall submit an analysis of the impacts of detained stormwater flows on downstream areas within the watershed. These impacts shall be identified with concurrence from the Municipal Engineer. The analysis shall include hydrologic and hydraulic calculations necessary to determine the impact peak discharge modifications from the proposed development have on critical locations such as dams, tributaries, existing developments, undersized culverts, floodprone areas, etc.

L. When stormwater management facilities are proposed within one thousand (1,000) feet of a downstream Municipality, the analysis of downstream impacts shall be submitted to the downstream municipality's engineers for review and comment.

Section 302. Sensitive Areas and Developments

Sensitive areas and water quality sensitive developments have been identified which require special consideration with regard to stormwater management.

Sensitive areas are defined as those areas that, if developed, have the potential to cause catastrophic loss to a Water Authority well field. These areas consist of the delineated 1-year zone of contribution and direct upslope areas tributary to the wells (see Appendix B, Exhibit 1). Municipalities may update the sensitive area boundaries based on new research or studies as required.

Water Quality Sensitive (WQS) developments are defined as a land development project that has a high potential to cause catastrophic loss to local water quality, and could potentially threaten ground water reservoirs. The following is a provisional list of water quality sensitive developments. This list may be amended at the discretion of the local municipality.

- Vehicle fueling stations
- Industrial manufacturing sites*
- Salvage yards
- Recycling centers
- Hazardous material storage areas*
- Interstate highways

* The Municipal Engineer will make the determination relative to what constitutes these classifications on a case-by-case basis. The Pennsylvania DEP wellhead protection contaminant source list shall be used as a guide in these determinations. Industrial manufacturing site and hazardous material storage areas must provide NPDES SIC codes.

Section 303. Performance Standards

A. **General** - Post-development rates of runoff from any regulated activity shall not exceed the peak release rates of runoff prior to development for the design storms specified.

B. **Sensitive Area District Boundaries** – The location of sensitive areas or sensitive area districts (SAD) within the watershed are illustrated on an official map, which is available for inspection at the municipal office. A reduced scale copy of this map is included as Exhibit 1 in Appendix B of this Ordinance. The exact location of the boundaries of sensitive areas are set by drainage areas tributary to each of the points of interest as illustrated in Appendix B. The exact location of these boundaries as they apply to a given development site, shall be determined using mapping at a scale which accurately defines the limits of the sensitive area. If the project site is within the sensitive area (in whole or in part), 2-foot contour interval mapping shall be provided to define the limits of the sensitive area. If the project site is adjacent to but within 500 linear feet of a defined Sensitive Area, a 5-foot contour interval map defining the limits of the Sensitive Area shall be included in the Stormwater Management Plan to document the site's location relative to the sensitive area.

C. **Sites Located in More Than One (1) District** - For a proposed development site which is traversed by a SAD boundary, the design criteria for sensitive areas must be applied if post-development runoff is directed towards the sensitive area.

D. **Off-Site Areas** - Off-site areas that drain from sensitive areas through a proposed development site that is located entirely in a non-sensitive area are not required to use or apply the sensitive area criteria.

E. **Site Areas** - Where the site area to be impacted by a proposed development activity differs significantly from the total site area, only the proposed impact area shall be subject to the design criteria.

F. **"Downstream Hydraulic Capacity Analysis"** - Any downstream or off-site hydraulic capacity analysis conducted in accordance with these standards shall use the following criteria for determining adequacy for accepting increased peak flow rates:

1. Natural or man-made channels or swales must be able to convey the post-development runoff associated with a 2-year return period event within their banks at velocities consistent with protection of the channels from erosion. Acceptable velocities shall be based upon criteria included in the DEP *Erosion and Sediment Pollution Control Program Manual*.

2. Natural or man-made channels or swales must be able to convey the post-development 25-year return period runoff without creating any hazard to persons or property.

3. Culverts, bridges, storm sewers or any other facilities which must pass or convey flows from the tributary area must be designed in accordance with DEP, Chapter 105 regulations (if applicable) and, at a minimum, pass the post-development 25-year return period runoff.

4. It must be demonstrated that the downstream conveyance channel, other stormwater facilities, roadways, or overland areas must be capable of safely conveying the 100-year design storm without causing damage to buildings or other infrastructure.

5. Where the downstream conveyance channel or other facility is located within a special flood hazard area (as documented on the municipal Flood Insurance Rate Map), it must be demonstrated that the limits of said flood hazard area are not increased by the proposed activity.

6. Stormwater management ponds that fall under the DEP Chapter 105 Criteria of a “Dam” must meet the criteria within Chapter 105.

Section 304. Calculation Methodologies

Design criteria and calculation methodologies have been classified by functional group for presentation as follows: 1) peak runoff rate discharge requirements; 2) stormwater pond capture volumes; 3) recharge volumes; 4) storm drain design including conveyance, channel protection, and stability; and 5) water quality standards.

These criteria and calculation methodologies have been developed to simplify stormwater management designs, unify methods, remove model parameter subjectivity, remove improperly used methods, and to ensure stormwater management decisions are based more realistically on hydrologic processes. In addition, common sense should always be used as a controlling criteria.

These standards provide consistent and process oriented design procedures for application by land development professionals. It is recognized that in an attempt to generalize the computational procedures, assumptions have been made which on some occasions may be violated. If such a violation is identified, alternate standards and procedures may be applied. Both the violation and the alternate procedures to be applied must be documented by a hydrologist or hydrogeologist. Any request for use of alternate standards or procedures under this provision must be agreed to by the local municipal engineer prior to formal submission of plans for consideration by the municipality.

A flow chart documenting the stormwater management design process is provided as Exhibit-2 in Appendix B of this ordinance.

A. Peak Runoff Rate Control

1.a. Any site where the increase in post-development peak runoff rates is determined to be negligible by the Municipal Engineer is exempt from the requirement to provide stormwater detention. In support of this exemption, it must be shown that the downstream conveyance systems have adequate capacity to convey the additional discharge without adversely affecting downstream properties. This does not exempt the requirement for implementation of designs for water quality, stormwater conveyance, and/or recharge as required. A Stormwater Management Plan and report documenting these design elements is also required. The Municipal Engineer shall use a 5% increase as a general benchmark for defining "negligible". The final definition of "negligible" shall be at the Municipal Engineer's discretion.

Prior to using this exemption (and prior to any land development plan submission), the Design Engineer must provide written documentation and computations as to why no peak runoff control should be required. The Municipal Engineer has the right to reject any plan which uses this assumption without prior approval of the Municipal Engineer. The intent of this exemption is to eliminate the need for multiple or "piggyback" detention facilities as a result of minor changes in imperviousness or land use upstream of existing stormwater control facilities.

1.b. Small sites (<5 acres) located directly adjacent to the main stem of creeks or within the floodplain are not required to provide stormwater detention unless directed to do so by the Municipal Engineer as a result of a documented drainage problem. All other stormwater management standards must be implemented including water quality, adequate stormwater conveyance, and/or recharge as required. The Municipal Engineer has the right to reject any plan that uses this exemption without prior approval of the Municipal Engineer.

2. Stormwater management analysis must be performed using the following models. The size criteria are based on drainage area size including site area and all off-site area draining across the development.

Up to 100 acres in size	NRCS's TR-55 or TR-20
Over 100 acres in size	NRCS's TR-20 or HEC-1 (HEC-HMS)

The Rational Method or Modified Rational Method may be used for any site less than or equal to two (2) acres in size without prior authorization from the Municipal Engineer. The Rational Method or Modified Rational Method may also be used for sites between two (2) and twenty (20) acres in size where the Municipal Engineer has approved the method's use. In this case the Design Engineer must make a written request to the Municipal Engineer explaining why the use of the Rational Method is more appropriate than the NRCS's methods for the site in question. The Design Engineer should keep in mind that the Rational Formula methodology was not calibrated to account for the karst nature of the Spring Creek Drainage Basin; and therefore, its use should be limited to the special cases identified above. In addition, since the minimum discharge criteria are based on a calibration of the NRCS runoff mode, their use is not appropriate if the Rational Method is used for runoff computations.

The Municipal Engineer has the right to reject any SWM design that uses hydrograph combinations with the Rational Method where the designer has not validated that the effects of the timing differences are negligible. In addition, the Municipal Engineer has the right to reject any SWM design that improperly uses the method for determining runoff volumes or does not properly apply the method.

More intensive physically based models may be used at the discretion of the Municipal Engineer, but only for sites greater than 100 acres in size.

Commercial software packages that use the basic computational methods of TR-55 or TR-20 are permitted.

The NRCS models and methods recommended above are based on data collected from actual watersheds. In contrast to this, stormwater management analysis for land development activities is often conducted using property lines to define drainage boundaries. Drainage areas based on property boundaries are not true watersheds and are referred to here as "hypothetical" drainage areas. It is known that these hypothetical drainage areas do not respond like natural watersheds. Peak runoff rates from hypothetical drainage areas are much smaller than comparable runoff rates from natural watersheds of the same size. Therefore, wherever possible, pre- and post-development stormwater analysis should be conducted for watersheds that are as nearly natural as possible. Also, conducting stormwater analysis for a lot by lot comparison, such as within residential developments is not permitted. Partitioning drainage areas into different sub-watersheds for the post-development scenarios is acceptable.

It is noted that natural watershed boundaries should not be used where the relative size of the watershed compared to the site size would inappropriately

distort the pre- to post-development runoff comparison. In these cases a hypothetical drainage area defined by the property boundary should be used because it will allow for a better estimate of runoff changes directly downstream of the site. In addition, the designer should recognize that, within the Spring Creek Watershed, typical hypothetical drainage areas, in their pre-development or natural condition, do not produce surface runoff during minor to moderate rainfall events. Available hydrologic models do not accurately reflect this condition. This often results in post-development nuisance flooding since the models over-estimate the pre-development runoff magnitude.

3. Major natural drainage divides may not be altered without the prior consent of the Municipal Engineer.

4. Pre- and post-development stormwater management analysis shall be conducted using the following design storms:

1 year	2-year
10-year	100-year

For sites less than one (1) acre in total area that connect directly to existing storm sewer systems, surface or subsurface (underground) stormwater detention facilities only need to be designed to control storm events up to the design return period of the existing pipes (usually 10 years). However, it must be demonstrated that adequate conveyance capacity (overland or within the existing storm sewer system) exists to ensure that flooding or damage from proposed releases will not exceed the existing potential for the system. If warranted by historic flooding in the tributary storm sewer system, the municipality may require more stringent criteria.

The Municipal Engineer may waive the requirement to detain the 100-year storm as long as the discharge is to a well defined, functioning conveyance system that does not currently exhibit flooding or other conveyance problems. The downstream conveyance system must be analyzed for the 100-year event to ensure that the proposed development will not increase flooding or damage to existing buildings and/or infrastructure.

5. The following 24-hour precipitation depths shall be used for stormwater management analysis for the entire Spring Creek Basin. These values override the use of TP-40 (the basis of the NRCS 24-hr precipitation maps).

Return Period	Precip. Depth
1-year	2.2 inches
2-year	2.6 inches
10-year	3.5 inches
25-year	4.2 inches
50-year	4.7 inches
100-year	5.3 inches

6. The NRCS's Type II precipitation distribution is required for all stormwater management analyses.

7. The NRCS's dimensionless unit hydrograph "k" factor shall be 484 for both pre- and post-development stormwater analyses.

8. All undeveloped areas are to be modeled as meadow or woods in good hydrologic condition. Existing impervious areas may be modeled as being impervious for pre-development conditions. The only exception is areas that are actively in agricultural production (non-fallow). The designer may model the watershed using the actual agricultural land use/cover condition to show that increases from the site as compared to the all meadow condition are negligible.

9. The NRCS's curve number (CN) shall be used as the rainfall to runoff transformation parameter for all stormwater management analyses.

10. Curve numbers should be rounded to tenths for use in pre-packaged hydrologic models. It should be recognized that the CN is only a design tool with a large degree of statistical variability. For large sites, CN's should realistically be rounded to the nearest whole number.

11. The NRCS's method to determine unconnected impervious area adjustments for CN can be used for distinctly defined impervious land areas that flow onto pervious areas in a dispersed manner. The method may only be used to calculate runoff from site impervious areas that actually flow across pervious areas. The method cannot be applied to the entire site using average weighted CN values.

12. Soils underlain by carbonate geology (limestone or dolomite) shall have a hydrologic soil group (HSG) B used for both pre- and post-development conditions regardless of the NRCS or Soil Survey's description, except for the following two conditions:

    a. Compacted structural fill areas shall use a minimum of HSG C for post development conditions regardless of the NRCS or Soil Survey's description. For most developments compacted structural fill areas are under impervious surfaces, but may include islands within parking areas, fringe land, etc. A HSG C shall also be applied to large projects that clear and compact building pad areas for later phases of development under an initial phase. The Municipal Engineer shall make the final determination as to what areas of a land development site constitute compacted structural fill. The intent is to account for large compacted areas, and not minor grading within lawn areas.

    b. Soils identified as "on flood plains" or "on terraces above flood plains" in the Centre County Soil Survey will use the HSG as designated in the Soil Survey. Refer to Appendix A for a list of the soils.

13. Soils not underlain by carbonate geology shall use the HSG as specified by the NRCS or Soil Survey's description, except for the following two conditions:

    a. Wooded areas on HSG C and D soils shall be treated as HSG B for pre-development conditions. Disturbed post-development wooded areas

shall carry the NRCS or Soil Survey's defined HSG with a minimum HSG of B.

b. Highly compacted structural fill areas shall use a minimum of HSG C for post-development conditions regardless of the NRCS or Soil Survey's description. For most developments these areas are normally covered with impervious surfaces, but may include islands within parking areas, fringe land, etc. A HSG of C shall also be used for large projects that clear and grade land for later phases of development. The Municipal Engineer shall make the final determination as to what areas of a land development site constitute compacted structural fill. The intent is to account for large compacted areas, and not minor grading within lawn areas or small areas around buildings, etc.

14. Areas draining to closed depressions must be modeled by removing the storage volume from the pre-development condition. The designer may assume that infiltration in the closed depression does not occur during a design runoff event. Areas draining to closed depressions may also be used to adjust peak runoff rates to stormwater management ponds for the post-development analysis. This allowance has been developed to entice designers to intentionally design or leave in place small closed depressions that can reduce the total volume required from a stormwater management pond. The site designer is responsible to document downstream impacts if the closed depression were removed.

15. Drainage areas tributary to sinkholes shall be excluded from the modeled point-of-interest drainage areas defining pre-development peak flows. Assumptions that sinkholes spill-over during some storm events must be supported by acceptable documentation (as determined by the Municipal Engineer). In addition, the design professional must be aware that bypassing or sealing sinkholes will frequently result in downstream flooding and should not be done if existing downstream flooding already occurs. The site designer is responsible to document downstream impacts if the sinkhole were to stop taking stormwater runoff.

16. Ponds or other permanent pools of water are to be modeled by the methods established in the NRCS's TR-55 manual (1986). However, more rigorous documented methods are acceptable (as determined by the Municipal Engineer).

17. The NRCS antecedent runoff condition II (ARC II, previously AMC II) must be used for all simulations. The use of continuous simulation models that vary the ARC are not permitted for stormwater management purposes. In addition, prior to any continuous simulation model being used in the Spring Creek Basin for any other purposes, the model unit hydrograph must be modified for common events in additional to extreme events based on an in depth analysis of historical data from the basin.

18. The following Time of Concentration (Tc) computational methodologies shall be used unless another method is pre-approved by the Municipal Engineer:

- Pre-development – NRCS's Lag Equation.

- Post-development; commercial, industrial, or other areas with large impervious areas (>20% impervious area) – NRCS's Segmental Method.

- Post-development; residential, cluster, or other low impact designs less than or equal to 20% impervious area – NCRS's Lag Equation.

The time of concentration is to represent the average condition that best reflects the hydrologic response of the area. For example, large impervious areas bordered by small pervious areas may not consider the effect of the pervious areas in the Tc computation. If the designer wants to consider the affect of the pervious area, runoff from the pervious and impervious areas must be computed separately with the hydrographs being combined to determine the total runoff from the area.

Under no circumstance will the post-development Tc be greater than the pre-development Tc for any watershed or sub-watershed modeling purposes. This includes when the designer has specifically used swales to reduce flow velocities. In the event that the designer believes that the post-development Tc is greater, it will still be set by default equal to the pre-development Tc for modeling purposes.

* Refer to item number 28 regarding impervious area flashing (IAF).

19. The following post-development minimum discharges are permitted for use with the NRCS (CN) runoff model*:

1-year return period	$Qp_{min} = 0.018\ (DA) + 0.2$
2-year return period	$Qp_{min} = 0.03\ (DA) + 0.4$
10-year return period	$Qp_{min} = 0.09\ (DA) + 1.0$

where: DA = the drainage area in acres
Qp_{min} = minimum allowable peak runoff rate in cfs

For return periods greater than 10 years, the minimum discharge shall be equal to the computed pre-development peak runoff rate.

The minimum discharge criteria above are not appropriate for use with the Rational Method. This is because these values were developed based on NRCS model corrections and do not actually represent a true physical process or discharge. However, common sense should be used by both the designer and reviewer in the evaluation of acceptable minimum discharges for use with the Rational Method.

The intent of the minimum discharge is to allow reasonable runoff release from a site when a hydrologic model has produced a pre-development runoff rate close to zero. The method is **NOT** permitted for areas that previously drained completely to sinkholes in order to bypass the sinkhole after development.

These minimum discharge values include the total of all stormwater management facilities discharges and undetained area discharges. Peak runoff rates for undetained fringe areas (where the designer has made a realistic effort to control

all new impervious areas) will be computed using the pre-development time of concentration for the drainage areas tributary to them. Undetained areas are those portions of the site that cannot be routed to a stormwater management facility due to topography and typically include lower pond berms, or small areas around entrance drives. The site drainage areas used shall represent the pre-development condition, even if drainage areas are altered following development.

20. All lined stormwater management ponds in carbonate and non-carbonate areas must be considered impervious and may not be used as pervious areas for stormwater management computations. "Lined" here means lined with synthetic liners or Bentonite. All other compacted soil liners will be considered to be HSG D for hydrologic computations.

21. Stormwater management ponds that have a capture depth for the purposes of water quality or volume capture shall assume a negligible discharge from these structures during design event routing. Only discharges from the primary principal spillway or emergency spillway need to be considered. Discharges from subsurface drains that tie into a principal spillway should not be considered during design event routing.

22. Stormwater management ponds that have a capture depth for the purposes of water quality or volume capture shall assume that the pond water quality or capture volume is full at the beginning of design event routing.

23. Stormwater management ponds must provide safe passage of the 100-year return period peak runoff rate assuming that all of the principal spillway orifices are fully clogged, and the principal spillway overflow is 50% clogged. A minimum of a 6-inch freeboard must also be maintained above the resulting "maximum" water surface elevations (W.S.E.). Any embankment emergency spillway can be assumed to be unclogged. SWM ponds with embankments completely made up of natural undisturbed soils (fully in “cut”) or where roadways act as the emergency spillway, are permitted. However, the Design Engineer must verify downstream stability and control.

24. All pre- and post-development comparisons of peak flows shall be rounded to tenths of a cfs. The intent here is to recognize the accuracy and precision limitations of hydrologic modeling procedures. Again, small differences between pre- and post-development discharge rates should be permitted when no negative downstream impacts will result

25. The full Modified Puls routing method must be used for stormwater management pond analyses. Simplified methods of determining pond size requirements such as those in TR-55 (1986) can only be used for preliminary pond size estimates. The full Modified Puls routing method must be used for stormwater management pond analyses. Simplified methods of determining pond size requirements such as those in TR-55 (1986) can only be used for preliminary pond size estimates.

26. Pre-packaged hydraulic programs are not approved for the analysis of underground stormwater management facilities unless it can be verified that the program rounding subroutines used for the stage/storage data do not affect the

results. This is because, for very small storage volumes, the program may round off the volume to a significant percentage.

27. Full supporting documentation must be provided for all stormwater management designs.

28. Designs must be checked for Impervious Area Flash (IAF). This check is used to determine if flooding may occur due to poor modeling choices specifically related to the time of concentration. This analysis requires that the watershed impervious area be modeled without the pervious areas. The time of concentration should also be determined from the impervious areas only. If the IAF analysis results in a higher peak runoff rate at a culvert or discharge from a pond, this higher rate must be used for the final design/comparison. The check will frequently yield higher values if a watershed's impervious area is located primarily near the watershed outlet or point of interest.

B. Pond Capture Volumes (Cv)

To minimize nuisance flooding from small precipitation events, a runoff capture volume is required for all stormwater management ponds that do not discharge directly to natural, well-defined (with bed and banks) perennial streams. In general, natural well-defined streams in the Spring Creek Basin are limited to those delineated as USGS perennial streams. This should be treated as a guideline and not a steadfast rule. The final determination is at the discretion of the Municipal Engineer. The pond capture volume is a volume of runoff that will be retained in a pond below the elevation of any free surface principal spillway orifice. No principal spillway orifice (except those connected to subsurface drains), regardless of how small, shall be below the pond elevation equivalent to this volume.

The Centre County Conservation District (CCCD) receives numerous complaints regarding ponds that are located at the downslope edge of a property that result in discharging runoff onto downstream properties in an uncontrolled manner or where no existing defined outlet channel exists. This is a very common problem in areas underlain by carbonate rock. These discharges can cause erosion and flooding downstream. While the Pond Capture volume is intended to minimize some of these negative effects, it cannot deter or reduce the impacts from poor design practices. Therefore, whenever possible, the CCCD recommends that the designer consider the downstream morphological changes that may occur and, when possible, consider constructing conveyance systems to a stable natural channel. In some cases this may require cooperation between land owners.

The capture volume is defined as a runoff depth of 0.25 inches from all impervious areas tributary to the stormwater management facility. This volume will be allowed to infiltrate, evaporate, or dewater from a subsurface drain system connected directly to the facility's principal spillway. Supporting computations that show that 90% of the capture volume can dewater in a maximum of 72 hours must be provided. For surface ponds, the maximum depth of ponding for the capture volume shall be three (3) feet (a health and safety precaution). However, in areas under karst influence, a limiting maximum ponding depth of eighteen (18) inches is recommended. Designers may always

increase the capture volume to a value greater than the identified standard as long as the ponding depth criteria are met.

To simplify computational requirements for design event analysis, designers do not need to calculate discharges from subsurface drains related to the capture volumes if the filter media is sand, or smaller than AASHTO 57 stone. The capture volume is to control runoff rates from impervious areas and is not related to water quality. However, pond designs that include a water quality volume that is greater than the required capture volume, are assumed to have also met the required capture volume as long as it dewaters as required.

Designs that rely on the natural infiltration of insitu soils must provide documentation supporting the infiltration rates used for analysis. Infiltration rates reported in the Soil Survey of Centre County or other published rates may be used at the discretion of the Municipal Engineer.

The pond capture volume should always be used when up-slope areas are developed where the pond's design creates a point discharge that did not previously exist.

Stormwater Management detention facilities that connect directly to storm drain pipe networks that discharge to natural well-defined channels do not require a capture volume.

C. Recharge Volumes (Rv)

The purpose of the recharge portion of the ordinance is two-fold. First, the recharge requirement is to mitigate the loss of groundwater recharge associated with the creation of impervious surfaces. In addition the recharge criteria is to mitigate the increase in runoff volume associated with the creation of impervious surfaces. This increase in runoff volume has significant impacts on downstream landowners. These impacts are most often exhibited in the form of increased nuisance flooding and channel or drainage-way erosion and instability. According to local Municipal Engineers and representatives of the Centre County Conservation District, these problems are of significant local concern. The magnitude of these problems increases with the percentage of impervious coverage created on a site.

Recharge mitigation shall be provided for runoff from all proposed impervious areas. The required recharge volume shall be computed as 0.5” of runoff from all proposed impervious areas. It is noted that lined detention ponds and compacted fill areas are considered to be impervious when calculating site impervious area for recharge considerations. In addition, land areas covered by paver blocks, pervious pavement, and other structural surface treatments which permit surface infiltration can be treated as pervious areas when calculating the site impervious area for recharge considerations as long as the structural infiltration practice is supported by sound design and appropriate construction specifications. The Municipal Engineer may require submission of supporting documentation prior to approving structural infiltration areas as pervious areas.

The following design practices can be used as credits to reduce the recharge volume requirement:

1. Residential Roof Areas (detached, duplex, and townhome dwellings) and commercial /industrial buildings with roof areas less than 5,000 square feet can be removed from the computed impervious area when these roof areas are sumped to dry wells designed in accordance with the following minimum standard:

   SUMP DESIGN CRITERIA: To meet the recharge criteria, sump storage or voids volume shall be equal to 0.04 cubic feet per square foot of roof area (0.5 inch rainfall depth). If sump stone has a voids ratio of 40%, the total sump volume will be 0.10 cubic feet per square foot of roof area. When designed only to meet this recharge criteria, the maximum size for a single sump is 100 cubic feet, and the minimum sump surface area (A) to depth (D) ratio (A/D) must be a minimum of 4/1. The sump depth less any freeboard should not exceed 24". This roof sump standard shall apply unless the municipality has a separate roof sump standard for water quantity or peak control.

2. All or portions of driveways, roadways, and parking areas can be removed from the impervious area calculation when sheet flow from these areas is directed to undisturbed natural buffer/ filter areas or constructed filter strips. This flow must be dispersed as sheet flow as it crosses the buffer / filter area. Sheet flow velocities should be non-erosive as they cross the impervious area / filter interface.

   To ensure proper infiltration characteristics the natural soil profile within natural buffer / filter areas can not be disturbed during construction. Completely undisturbed natural recharge areas serve this function best. However, minor surface scaring, seeding, and landscaping is permitted in these areas as long as natural grades are not altered. In special cases, when approved by the Municipal Engineer, minor grading, combined with soil profile reconstruction may be permitted in natural buffer / filter areas. In addition, the following standards apply to **natural filter / buffer areas**:

   a. Natural filter / buffer areas must have a minimum width of five (5) feet or one-half of the impervious area drainage length immediately tributary to the buffer area, whichever is greater. This width is measured parallel to the direction of sheet flow.

   b. To qualify for a recharge volume credit, the surface slope of natural filter / buffer areas must be conducive to recharge, and not result in flow concentration or erosion. To meet this intent, the surface slope of the area tributary to the natural buffer/filter area, and the surface slope of the natural buffer/filter area itself may not exceed 5%. In special cases steeper slopes may be used if specifically authorized by the Municipal Engineer.

   c. The total impervious area tributary to a natural buffer / filter area can not exceed twice the buffer / filter area.

   To qualify for a recharge volume credit, **constructed filter strips** shall be designed to the following standards:

a. The minimum filter strip width shall be five (5) feet or one-half of the impervious area drainage length immediately tributary to the constructed filter strip, whichever is greater. This width is measured parallel to the direction of sheet flow.

b. The total impervious area tributary to a constructed filter strip area can not exceed twice the constructed filter strip area.

c. The surface slope of the area tributary to the constructed filter strip area, and the surface slope of the constructed filter strip area itself may not exceed 5% and 3% respectively. In special cases steeper slopes may be used if specifically authorized by the Municipal Engineer.

d. The filter strip surface shall consist of a minimum of six (6) inches of natural or reconstructed topsoil with a stable grass surface treatment. Reconstructed topsoil designs must be approved by the Municipal Engineer prior to application. Reconstructed topsoil consists of soils augmented by tillage and the addition of soil amendments such as compost, lime, animal manures, crop residues, etc.

e. To minimize erosion of the topsoil layer during construction, it is recommended that these areas be sodded. However, the Municipal Engineer may permit the use of an acceptable erosion control seeding application. In this later case, any loss of topsoil and seed must be replaced until a permanent vegetative stand is achieved.

3. Sidewalks separated from roadways and / or other impervious surfaces by a grass strip of equal or greater width than the sidewalk itself can be removed from the impervious area calculation when the sidewalks are graded so that sheet flow from the walk is directed to the grass strip. Sidewalks with steep longitudinal slopes that themselves would act as channels during runoff events can not take advantage of this credit. A five percent (5%) longitudinal sidewalk slope shall be used as the benchmark defining steep slopes.

4. Impervious areas tributary to natural closed depressions can be subtracted from the total site impervious area used in the recharge volume calculation as long as a qualified geotechnical engineer or soil scientist certifies to the soundness of these site specific applications. Water quality pre-treatment may be necessary prior to the direct discharge of runoff to existing closed depressions or sinkholes.

5. Impervious areas tributary to man-made closed depressions can be subtracted from the total site impervious area as long as a qualified geotechnical engineer or soil scientist certifies to the soundness of these site-specific applications. Man-made closed depressions can be created through the use of low head berms 1 foot or less in height.

6. The entire capture volume provided in a pond without a subsurface drain may be used as a credit towards the recharge volume requirement.

7. Fifty percent (50%) of the capture volume in a pond that includes a subsurface drain may be credited towards the recharge volume requirement.

8. Additional credits may apply for undisturbed land areas that are known to have high infiltration capacity and that are maintained or enhanced. These areas must be defined and quantified from actual site data collection.

After credits, the remaining recharge volume shall be directed to a Recharge BMP such as infiltration trenches, beds, etc. These facilities can be located in open areas or under pavement structures. The appropriateness of the particular infiltration practice proposed, as well as the design parameters used, shall be supported by a geotechnical report certified by a qualified professional (soil scientist, geologist, hydrogeologist, geotechnical engineer, etc.).

Stormwater recharge requirements or credits affect stormwater management design requirements. For stormwater management computations, the reduction of site CNs based only on a weighting type analysis, as is sometimes done for cluster type developments, is not permitted. However, for stormwater management purposes, the CN for recharged areas can be computed using the NRCS method for disconnected impervious areas. The actual hydrologic process that occurs within the basin must be stressed in all recharge situations.

These recharge requirements must be met on all sites unless it can be demonstrated that recharge would be inappropriate. Any request for such a waiver from these recharge requirements must be accompanied by a supporting report certified by a qualified professional (soil scientist, geologist, hydrogeologist, geotechnical engineer, etc.).

Developers and site design professionals are encouraged to use a higher standard for recharge volume on sites where local site conditions do not restrict a higher standard.

Water Quality Sensitive (WQS) developments must use an acceptable pre-treatment BMP prior to recharge. Acceptable pre-treated BMPs for these developments include BMPs that are based on filtering, settling, or chemical reaction processes such as chemical coagulation.

Accounting for recharge within lined stormwater management ponds is not permitted. However, if unlined, uncompacted ponds and/or depressed lawn areas are used to satisfy water quality or capture volume criteria, these areas and volumes can also be used to meet recharge requirements as previously defined. Additional recharge volume may be credited to these areas as long as it is demonstrated by a qualified professional that recharge processes can naturally occur in these areas.

Finally, because this analysis is concerned with trying to adequately represent real processes that occur within the Watershed, there will be areas that cannot physically recharge stormwater. These areas include exfiltration areas that are commonly found at the base of wooded hillsides where clay pans exist, and saturation areas near major streams or floodplains. These areas may not accept recharge during most runoff events. These areas are exempt from recharge requirements when these conditions are documented and certified by a qualified professional (soil scientist, geologist,

hydrogeologist, or geotechnical engineer). In addition, stormwater management techniques relying on infiltration techniques are not permitted in these areas.

The Municipal Engineer may waive the recharge requirement in the following situations:

1. The Municipal Engineer may waive the recharge requirement in highly developed areas or areas undergoing redevelopment where the Municipal Engineer has determined that forced recharge could have adverse impacts on adjacent landowner structures, property, or municipal infrastructure. These waivers should be limited to small land areas (generally less than 5 acres in size), where the ability to place recharge beds may be limited or may hinder redevelopment.

2. The Municipal Engineer may waive the recharge requirement in areas where a qualified soils scientist or geologist has determined that none of the site soils are suitable for recharge, or that the location of the suitable soils is such that harm to adjoining properties could occur as stated under item 1 above.

3. The Municipal Engineer may waive the recharge requirement in areas where recharge can not physically occur as documented by a qualified soil scientist, geologist, or hydrologist. These areas include:

    a. Exfiltration areas commonly found at the base of wooded hillsides where clay pans or fragipans exist; and

    b. Saturation areas near major streams or floodplains.

As identified above, recharge analysis and/or waiver requests must be supported by a geotechnical report sealed by a qualified professional (soil scientist, geologist, hydrogeologist, or geotechnical engineer). The intent of this report will be to establish the suitability of a particular parcel of land or area for recharge, and to identify areas on a development site appropriate for recharge. It is recommended that the geotechnical / soils consultant discuss the extent and approach to the analysis with the Municipal Engineer prior to initiating the field investigation. At a minimum this report should include the following information:

1. A description of the geotechnical site investigation performed including the methods and procedures used;

2. Data presentation;

3. Analysis results including the following minimum information:

    a. A map identifying site areas inappropriate for recharge along with supporting justification. In addition to illustrating topographic features, significant geologic and hydrologic features should be identified (rock outcrops, sinkholes, closed depressions, etc.

    b. Determination of the permeability coefficient for potential recharge areas.

    c. Determination of the infiltration capacity of natural site soils.

d. Location, depth, and permeability coefficient for any restrictive layers identified.

e. Soil uniformity.

f. Depth to bedrock in potential recharge areas, and a statement reflecting the uniformity of the depth to bedrock across the site.

g. A statement relating to the site's proximity to fracture zones within the bedrock.

h. Additional information deemed pertinent by the geotechnical engineer.

4. Recommendations for any special design considerations necessary for the design of recharge systems on the site. For example, required soil depth over bedrock, appropriate surface grades over recharge areas, appropriate hydraulic head over recharge areas, etc.

5. Justification as to why the site should be developed to a high impervious density if the site has adverse soil and geotechnical limitations, which prohibit the ability to induce natural recharge. Explain how these limitations will not create the potential for undue harm to the environment and the Spring Creek Watershed when the site is developed.

The following guidelines are provided relative to the use of subsurface exfiltration BMP's (often incorrectly referred to as engineered infiltration BMPs):

1. Soils should have a minimum percolation rate of 50 min/cm for effective operation of subsurface exfiltration BMPs. If no site soils have percolation rates of 50 min/cm, subsurface exfiltration BMPs should not be used.

2. A minimum of 30 inches of soil must be maintained between the bottom of a subsurface exfiltration BMP and the top of bedrock or seasonally high groundwater table. This statement is subject to the recommendation of a qualified Geotechnical Engineer.

3. If the minimum percolation rate is not met and/or the minimum soil depth can not be maintained on a site, recharge should be accommodated by directing shallow sheet flow from impervious areas across surface filter strips and/or undisturbed natural areas, or some other innovative surface infiltration feature should be used. Limiting subsurface percolation rates and/or depth to bedrock shall not by themselves warrant a recharge waiver.

In addition, since recharge is intended as a volume control, innovative or new methods that address the significant increase in the volume of runoff from sites having large impervious areas are encouraged These volume control alternatives can be used only if they can be shown to function with the original intent through sound engineering and science. The final determination of "original intent" shall always be the right of the Municipal Engineer.

D. Storm Drain Conveyance System Design

Storm drainage conveyance systems consist of storm sewer pipes, swales, and open channels. Computational methods for design of storm drain conveyance systems shall be as follows:

1. Recommended computational methods (models) for storm drain design are based on site or watershed drainage area as follows:

Up to 200 acres in size	Rational Method
Between 200 acres and 1.5 Square miles	HEC-1 PSRM TR-20
Over 1.5 square miles in size	PSU-IV with the carbonate adjustment factor at the discretion of the Municipal Engineer

Other methods as approved by the Municipal Engineer such as SWMM, SWIRM-ROUTE, etc.

2. Rational Coefficients used are to be from Rawls et al. (1981), PA DOT Design Manual 2-10 or using the Aron curves to convert CNs to C. If the Aron curves are used, all CNs must be applicable to the HSG as identified by the NRCS.

   The Design Engineer may choose to use the following Rational C coefficients without regard to soil HSG for small sites. However, it is recommended that they be used only for storm drains up to 24" in diameter. The use of these conservative values shall fully be the choice of the Design Engineer.

   All impervious areas: C = 0.95
   All pervious areas: C = 0.30

3. Storm drains shall be designed at a minimum using a 10-year runoff event without surcharging inlets. Storm drains tributary to a multiple site SWM facility across municipal roads or crossing other properties must convey, at a minimum, a 25-year runoff event without surcharging inlets. Runoff events in excess of the indicated design event must be conveyed safely downstream.

4. Inlets on grade cannot assume a sumped condition for hydraulic modeling (i.e., top of inlet casting set below pavement surface in parking areas).

5. The Municipal Engineer may require the analysis of the 100-year peak runoff rates for conveyance purposes in some instances where regional SWM facilities are employed.

6. Any storm drain within State or Federal rights-of-ways or that falls under the design criteria of any higher authority must meet the requirements of that agency in addition to the minimum requirements of this ordinance.

7. The time of concentration (Tc) can be computed by any method which best represents the subject watershed. However, the NRCS's segmental method is

not recommended for use with drainage areas that are predominately undeveloped and are greater than 100 acres in size. The NRCS Lag Equation or another more appropriate method should be used under these conditions.

8. For any drainage area smaller than 5 acres in size, a Tc of 5 minutes may always be assumed at the discretion of the Design Engineer (for the post-development condition), without needing to provide supporting documentation.

9. Precipitation values applicable to the entire Spring Creek Drainage Basin are those reflected in the PA DOT's IDF curves for Region 2, regardless if the area was formerly considered in Region 3.

10. Storm drain conveyance system stability (swales, open channels, and pipe discharge aprons) shall be computed using a 10-year return period peak runoff rate.

11. Storm sewers, where required by zoning and land use densities, shall be placed under or immediately adjacent to the roadway side of the curb, or as directed by the Municipality, when parallel to the street within the right-of-way.

12. When located in undedicated land, they shall be placed within a drainage easement not less than twenty (20) feet wide as approved by the Municipal Engineer.

13. The use of properly designed, graded and turfed drainage swales is encouraged in lieu of storm sewers in commercial and industrial areas and, where approved by the Municipal Engineer, in residential areas.

    Such swales shall be designed not only to carry the required discharge without excessive erosion, but also to increase the time of concentration, reduce the peak discharge and velocity, and permit the water to percolate into the soil, where appropriate.

14. Inlet types and inlet assemblies shall conform to the Pennsylvania Department of Transportation Standards for Roadway Construction as approved by the Municipal Engineer.

    a. Inlets shall, at a minimum, be located at the lowest point of street intersections to intercept the stormwater before it reaches pedestrian crossing; or at sag points of vertical curves in the street alignment which provide a natural point of ponding of surface stormwater.

    b. Where the Municipality deems it necessary because of special land requirements, special inlets may be approved.

    c. The interval between inlets collecting stormwater runoff shall be determined in accordance with DM-2, Chapter 10, Section 5, "Capacity of Waterway Areas".

    In curbed sections, the maximum encroachment of water on the roadway pavement shall not exceed half of a through traffic lane or one (1) inch less than

the depth of curb during the 10-year design storm of five (5) minute duration. Inlets shall be provided to control the encroachment of water on the pavement. When inlets are used in a storm system within the right-of-way limits of a street in lieu of manholes, the spacing of such inlets shall not exceed the maximum distance of four hundred fifty (450) feet.

15. Accessible drainage structures shall be located on a continuous storm sewer system at all vertical dislocations, at all locations where a transition in storm sewer pipe sizing is required, at all vertical and horizontal angle points exceeding five (5) degrees, and at all points of convergence of two or more influent storm sewer mains. The construction locations of accessible drainage structures shall be as indicated on the land development drainage plan or area drainage plan approved by the Municipality.

16. When evidence available to the Municipality indicates that existing storm sewers have sufficient capacity as determined by hydrograph summation and are accessible, the subdivider may connect their stormwater facilities to the existing storm sewers so long as the peak rate of discharge does not exceed the amount permitted by this Ordinance.

17. All other storm drain design methods are to be the same as specified in existing local ordinances.

18. Computational procedures other than those indicated here should follow the methods of the Federal Highway Administration's Urban Drainage Design Manual [Hydraulic Engineering Circular No 22. (HEC-22)].

E. Water Quality Standards

Water Quality Performance Standards

To minimize adverse impacts to stream health resulting from stormwater non-point source (NPS) pollution, standards are provided for the implementation of Water Quality Best Management Practices (BMPs) to reduce NPS pollutant loadings resulting from land development activities. The following performance standards and guidelines shall be addressed at all sites where stormwater management is required.

1. Site designs shall minimize the generation of stormwater runoff through the use of low-impact design techniques.

2. Stormwater runoff from all land development activities should be treated through the use of non-structural and structural BMPs to effectively treat the adverse impacts of stormwater runoff including NPS pollutants.

3. Water Quality BMPs shall be incorporated into site designs to treat the required Water Quality volume as defined below.

4. The use of non-structural BMPs shall always take priority over the use of structural BMPs. The use of innovative BMPs and low-impact site planning is

encouraged to reduce the generation of stormwater runoff and effectively treat pollutants transported in stormwater from the site.

5. The use of multiple non-structural water quality techniques along with new, emerging, and innovative techniques is encouraged to improve the quality of stormwater runoff to receiving areas and reduce and/or eliminate the need for structural BMPs. The Municipal Engineer should be consulted to clarify the design concept for meeting or exceeding the intent of this section

6. Where non-structural BMPs are unable to effectively treat all of the stormwater runoff generated from land development activities, structural BMPs shall be designed to capture and treat the computed water quality volume (WQ_v).

7. The priority pollutant source areas to be treated with BMPs are streets, parking lots, driveways, and roof areas.

8. Due to the karst nature of the Watershed, stormwater discharges from water quality sensitive developments and discharges to sensitive wellhead protection areas (defined in Appendix B, Exhibit-1) will require special consideration. In these instances the applicant shall provide water quality pre-treatment (use of a filtering BMP and/or special structural design features) to prevent the discharge of stormwater contaminants to groundwater resources. In addition, hydrogeologic studies may be required to document potential karst related impacts.

9. Prior to stormwater management and water quality design, applicants should consult with the Municipal Engineer to verify stormwater quality criteria and present proposed features and concepts for the treatment of stormwater runoff. Following this meeting, the Municipal Engineer shall define any needed support studies or documentation.

Water Quality Volume (WQv)

The required water quality volume that must be treated for **non-sensitive areas underlain by carbonate rock** (see exhibits in Appendix B) within the Spring Creek Basin shall be computed as:

$$WQ_{depth} = 0.25+(0.012)2.9^{(0.044(SIA))}$$

$$WQ_v = WQ_{depth}(A)/12$$

Where:
- WQ_v = water quality volume in acre-feet
- WQ_{depth} = depth in inches that must be captured for impervious areas
- SIA = percent of site impervious area (all paved areas and roof with asphalt-based roofs)
- A = total of all paved areas and asphalt-based roofs on site in acres

The required water quality volume that must be treated **for any WQS development, on sites in sensitive areas underlain by carbonate rock, and all areas not underlain by carbonate rock** is to be computed within the entire Spring Creek Basin as:

$$WQ_{depth} = \text{the larger of 0.5 inches or } 0.25+(0.012)2.9^{(0.044(SIA))}$$

$$WQ_v = WQ_{depth}(A)/12$$

Where:
- WQ_v = water quality volume in acre-feet
- WQ_{depth} = depth in inches that must be captured for impervious areas
- SIA = percent of site impervious area (all paved areas and roof with asphalt-based roofs)
- A = total of all paved areas and asphalt-based roofs on site in acres

For designs in which the final roof material is unknown, the Design Engineer must assume an asphalt-based roof.

The water quality volume must be captured and treated through a water quality BMP over an extended period of time as per the specific requirements of each structure. Credits to reduce the effective impervious area are applicable as presented in Chapter 4 of the Stormwater Management Plan.

Water Quality Credits

Due to the karst nature of the Spring Creek Basin, the non-structural water quality credits and techniques identified below may be limited for suitability and use based on development type and location. These limitations for use are specified in the restrictions section for each credit. The Municipal Engineer may require additional documentation or investigation prior to use of each specific credit to reduce the risks of sinkhole development or groundwater contamination for sensitive areas and development types. No area may be double counted for use with credits. The combined credits of natural area conservation and vegetated filter strips is limited to 50% of the site's impervious area. The drainageway credit is limited to 50% of the site's impervious area. The drainageway protection credit is limited to 50% of the site's impervious area. The maximum total water quality credit for any site may therefore be 100% of the site's impervious area.

Non-Structural Technique	Water Quality Credit
Drainage-way Protection (DWP)	Subtract Drainage-way Protection Areas from impervious site area in WQ_v computation.
Natural Area Conservation (NAC)	Subtract Conserved Natural Areas from impervious site area in WQ_v computation.
Filter / Buffer Area	Subtract impervious areas discharged over pervious areas from impervious site area in WQ_v computation.

Drainage-way Protection

A water quality credit is given for the protection of natural drainage-ways on a development site. Natural karst drainage-ways within the Spring Creek Watershed often do not exhibit a defined channel bed and banks. More often, these drainage-ways appear as wide, shallow parabolic swales. These drainage-ways are an integral part of the natural drainage system, and often exhibit significant infiltration capacity. Protection of these drainage-ways is critically important to the health of the watershed.

The drainage-way protection (DWP) area is defined as an area centered on the drainage-way and having a minimum width of 300 feet. The Municipal Engineer may modify the defined minimum width in cases where natural land forms define an appropriate alternate width.

The impervious area used in the WQ_v equation for the development site may be reduced by twice the area of the preserved drainage-way (2 to 1 ratio).

1. Restrictions on the Credit:

   - Drainage-way protection areas must remain in an undisturbed condition during and after construction activities. There can be no construction activity within these areas including temporary access roads or storage of equipment or materials. Temporary access for the construction of utilities crossing this protection area may be permitted at the Municipal Engineer's discretion. However, the alignment of any such crossing must be perpendicular to the drainage-way.

   - These areas should be placed in a conservation easement or be permanently preserved through a similarly enforceable agreement with the municipality.

   - The limits of the undisturbed DWP area and conservation easement must be shown on all construction plans.

   - The DWP area must be located on the development site.

   - The maximum total DWPA credit is 100% of the site impervious area.

   - Water quality credits are not permitted for Water Quality Sensitive (WQS) developments.

2. Sensitive Area and Development Restrictions:

   - DWP areas may not be counted as a credit in sensitive areas unless the impervious area actually flows across the area as sheet flow.

   - Untreated urban runoff from sensitive development types may not be directed to DWP areas without pretreatment.

Natural Area Conservation

A water quality credit is given for natural areas that are conserved at the development site, thereby maintaining pre-development water quality characteristics. The impervious area used in the WQ_v equation for the development site may be reduced by the natural area conserved in the water quality volume computations. Natural area conservation is different than vegetated filter strip/recharge area and drainageway protection in that in some cases surface runoff may never be directed over the natural area (i.e., if upslope wooded areas are conserved).

1. Restrictions on the Credit:

   - Natural areas must remain in an undisturbed condition during and after construction activities. Temporary incidental land disturbance activities associated with utility construction may be permitted within the conservation area.

   - These areas should be placed in a conservation easement or similarly enforceable agreement with the municipality.

   - The limits of the undisturbed area and conservation easement must be shown on all construction plans.

   - The area must be located on the development site.

   - Water quality credits are not permitted for Water Quality Sensitive (WQS) developments.

   - The maximum total NAC credit is 50% of the site impervious area. However, the combination of NAC VFRS is also 50%.

2. Sensitive Area and Development Restrictions:

   - NAC areas may not be counted as a credit in sensitive areas unless the impervious area actually flows across the area as sheet flow.

   - Untreated urban runoff from sensitive development types may not be directed to natural areas without pretreatment.

Filter / Buffer Area

A water quality credit is given when stormwater runoff is effectively treated via a filter / buffer area or strip. A filter / buffer area is a vegetated boundary characterized by uniform mild slopes. Filter strips may be forested or vegetated with turf grass. Effective treatment is achieved when impervious area runoff is directed as sheet flow across vegetative filter or buffer areas (i.e., concentrated flow discharged to a filter strip does not meet water quality reduction criteria).

The area draining via overland sheet flow to an undisturbed, natural, vegetated filter strip (natural unmaintained meadow or forested area) can be subtracted from the site

impervious area (IA) on a 1:1 area ratio in the water quality volume computation. Impervious areas draining across constructed (disturbed or regarded) pervious areas can be subtracted from the site impervious area (IA) on a 1:1/2 area ratio in the water quality volume computation.

1. Restrictions on the Credit:

- The maximum impervious area that can be included in this credit, shall be computed as follows:

$$IA_c = W_{IA} \; L_{IA}$$

Where: IA_c = Impervious area recharge credit (L^2).
L_{IA} = Length of impervious area measured perpendicular to the sheet flow direction (L).
W_{IA} = Width of impervious area (L). Maximum width permitted for credit is the smaller of 100 feet or twice the width of the vegetated filter strip.

- To qualify for a water quality credit, natural and constructed filter areas or strips must meet the same restrictions identified for natural or constructed recharge areas with regard to width, length, slope, tributary drainage length, and construction. These restrictions are presented in Chapter 3.

- Runoff shall enter the filter / buffer strip as overland sheet flow.

- Filter/ buffer areas shall remain undisturbed/unmanaged other than to remove accumulated trash and debris.

- Water quality credits are not permitted for Water Quality Sensitive (WQS) developments.

- The maximum total water quality credit for vegetative filter / buffer areas is 50% of the site impervious area. However, the combination of NAC and filter / buffer areas is also 50%.

2. Sensitive Area and Development Restrictions:

- Untreated urban runoff from WQS developments may not be directed to filter / buffer areas without pretreatment.

Comments Related to Water Quality Credits

Concurrence of the Municipal Engineer is required prior to the use of all water quality credits for the reduction of the water quality treatment volume. The Municipal Engineer may approve the use of additional credits based upon sufficient documentation regarding suitability for sensitive development types and areas, pollutant removal effectiveness, and maintenance criteria. Multiple water quality credits cannot be claimed for the identical area of the site (i.e., a stream buffer credit and disconnecting roof recharge area cannot both be claimed for the same area).

Additional impervious coverage reduction using low impact development techniques (development practices which reduce the impact of urban runoff such as narrower residential road sections, smaller cul-de-sacs, smaller parking stalls, smaller building set-backs to reduce driveway lengths, etc.) will also reduce the required water quality treatment volume. Many of these techniques require prior approval by the municipality before implementation into land development design.

Section 305. Erosion and Sedimentation Requirements

A. Whenever the vegetation and topography are to be disturbed, such activity must be in conformance with Chapter 102, Title 25, Rules and Regulations, Part I, Commonwealth of Pennsylvania, Department of Environmental Protection, Subpart C, protection of Natural Resources, Article II, Water Resources, Chapter 102, "Erosion Control," and in accordance with the Centre County Conservation District and the standards and specifications of the appropriate municipal government.

B. Additional erosion and sedimentation control design standards and criteria that must be or are recommended to be applied where recharge or water quality BMPs are proposed and include the following:

1. Areas proposed for these BMPs shall be protected from sedimentation and compaction during the construction phase, so as to maintain their maximum infiltration capacity.

2. These BMPs shall not be constructed nor receive runoff until the entire contributory drainage area to the BMP has received final stabilization.

C. Adequate erosion protection shall be provided along all open channels and at all points of discharge.

Section 306. Sinkhole Protection

The use of sinkholes for stormwater management must be carefully planned, because discharging runoff directly into existing sinkholes is not an engineered stormwater solution. Aside from potential water quality effects, cover collapse sinkholes that exist throughout the watershed can be unstable, and it should be assumed that they could stop taking water at any time. Numerous sinkholes throughout the region already flood during larger runoff events. Nonetheless, in the watershed there are large drainage areas that completely drain to existing sinkholes and all upslope development tributary to them cannot be realistically stopped. Therefore the following sections have been developed.

A. Stormwater from roadways, parking lots, storm sewers, roof drains, or other concentrated runoff paths shall not be discharged directly into sinkholes without prior filtration in accordance with Section 308, B, below.

B. Sinkholes capable of absorbing substantial amounts of stormwater shall be protected by diverting such runoff around the sinkhole (refer to 306.F) or, upon recommended approval of the Municipal Engineer, by planting and maintaining a dense filter path of

suitable vegetative material in such a manner and location to disperse and slow the runoff to a sheet flow condition to promote the maximum possible filtration and sedimentation of impurities.

The filter path must be at least one hundred (100) feet in length and twenty (20) feet in width. Ten-foot wide filter paths are acceptable if land slope is less than two (2) percent.

Filter paths shall be designed and installed so that they filter sheet flow rather than concentrated flow. If concentrated flow occurs, grading and shaping or the use of best management practices such as grass waterways or drop structures may be required.

Sedimentation basins designed to DEP Chapter 102 Standards or permanent stormwater storage criteria, whichever is larger, and proposed vegetative filter paths, in conjunction with temporary stone filter check dams, shall be installed prior to subdivision or land development construction activities, where sinkholes are used to accept stormwater discharges.

C. If increased runoff is to be discharged into a sinkhole, even in filtered conditions, a hydrogeologic assessment of the effects of such runoff on the increased risk of land subsidence and adverse impacts to existing sinkhole flood plains and groundwater quality shall be made by a qualified professional and submitted with the stormwater management plan. Such discharge shall be prohibited if the Municipal Engineer determines that such poses a hazard to life, property or groundwater resources.

D. All sinkholes shall be posted by permanent on-site notices clearly visible at the sinkhole prohibiting any disposal of refuse, rubbish, hazardous wastes, organic matter or soil into the sinkhole. Rockfill may be permitted in the sinkhole for the purpose of preventing dumping of said materials.

E. To protect sensitive Karst areas, the Municipal Engineer may require basins to contain an impervious liner. The liner may be of the impervious membrane type, placed in accordance with the manufacturer's recommendations, or may be constructed by mixing Bentonite, or an approved alternative, with existing soil available at the site as approved by the Municipal Engineer.

F. If it is determined that runoff from upslope developing areas should be diverted around a sinkhole due to existing problems, the Municipal Engineer may require additional upstream volume controls as required to protect downstream areas.

Section 307. Design Criteria for Stormwater Management Facilities

Materials, Workmanship and Methods: All materials, workmanship and methods of work shall comply at a minimum with the Pennsylvania Department of Transportation Form 408 specifications, as accepted and commonly used by the respective Municipality, and shall be considered to be incorporated into this article as if copied in full. In the event a conflict arises between the requirements of this article and the Form 408 Specifications, the Municipal Engineer shall resolve the difference, and his opinion shall be binding.

A. General

1. Facilities in State Right-of-Ways – Any stormwater facility located on tate highway rights-of-way shall be subject to approval by the Pennsylvania Department of Transportation (PA DOT). Any stormwater facility that discharges directly onto state highway rights-of-way shall be subject to review by the PA DOT.

2. Water Obstructions – Any facilities that constitute water obstructions (e.g., culverts, bridges, outfalls, or stream enclosures), and any work involving wetlands as directed in PA DEP Chapter 105 regulations (as amended or replaced from time-to-time by PA DEP), shall be designed in accordance with Chapter 105 and will require a permit from PA DEP. Any other drainage conveyance facility that does not fall under Chapter 105 regulations must be able to convey, without damage to the drainage structure or roadway, runoff from the 25-year design storm with a minimum of 1.0-foot of freeboard measured below the lowest point along the top of the roadway. Roadway crossings located within designated floodplain areas must be able to convey runoff from a 100-year design storm with a minimum of 1.0-foot of freeboard measured below the lowest point along the top of roadway. Any facility that constitutes a dam as defined in PA DEP Chapter 105 regulations may require a permit under dam safety regulations. Any facility located within a PA DOT right-of-way must meet PA DOT minimum design standards and permit submission requirements.

3. Conveyance Facilities – Any drainage conveyance facility and/or channel that does not fall under Chapter 105 Regulations, must be able to convey, without damage to the drainage structure or roadway, runoff from the return period design storm as specified in Section 311, A. Conveyance facilities to or exiting from stormwater management facilities (i.e., detention basins) shall be designed to convey the design flow to or from that structure. Roadway crossings located within designated floodplain areas must be able to convey runoff from a 100-year design storm. Any facility located within a PA DOT right-of-way must meet PA DOT minimum design standards and permit submission requirements.

B. Stormwater Basin Design Considerations

Stormwater management basins for the control of stormwater peak discharges shall meet the following minimum requirements.

1. The design of all facilities over limestone formations shall include measures to prevent groundwater contamination and where required, sinkhole formation. Soils used for the construction of basins shall have moderate to low erodibility factors (i.e., "K" factors of 0.32 or less). Any basin greater than 4 feet in height, measured from the top of berm to the downslope toe of the abutment, must also contain:

    a. Berm soil specifications;

    b. A determination if site soils are available for the construction of the berm or cutoff trench;

c. An impervious cutoff trench, which extends the full length of the downstream berm located in fill.

2. Energy dissipators and/or level spreaders shall be installed at points where pipes or drainageways discharge to or from basins. Generally, outlet pipes designed to carry the pre-development, 1-year storm flow will be permitted to discharge to a stream with only an energy dissipator. Discharges to drainage swales shall be spread with a level spreader or piped to an acceptable point.

3. Outlet structures:

   a. Outlet structures within detention/retention basins shall be constructed of reinforced concrete or an approved alternate. With the exception of those openings designed to carry perennial stream flows, design openings shall have childproof, non-clogging trash racks over all openings nine inches (9") or smaller in any dimension. Outlet aprons shall be designed and shall extend at a minimum to the toe of the basin slope. Where spillways will be used to control peak discharges in excess of the 10-year storm, the control weirs shall be constructed to withstand the pressures of impounded waters and convey flows at computed outlet velocities without erosion.

   b. All metal risers, where approved for use, shall be suitably coated to prevent corrosion. A trash rack or similar appurtenance shall be provided to prevent debris from entering the riser. All metal risers shall have a concrete base attached with a watertight connection. The base shall be sufficient weight to prevent flotation of the riser. An anti-vortex device, consisting of a thin vertical plate normal to the basin berm, shall be provided on the top of all metal risers.

4. Emergency Spillway:

   a. Any stormwater management facility (i.e., detention basin) designed to store runoff and requiring a berm or earthen embankment required or regulated by this Ordinance shall be designed to provide an emergency spillway to handle flow up to and including the 100-year post-development conditions. The height of embankment must be set as to provide a minimum 0.5 foot of freeboard above the elevation required to safely pass the 100-year post-development inflow. Should any stormwater management facility require a dam safety permit under PA DEP Chapter 105, the facility shall be designed in accordance with Chapter 105 and meet the regulations of Chapter 105 concerning dam safety which may be required to pass storms larger than a 100-year event.

      Any underground stormwater management facility (pipe storage systems) must have a method to bypass flows higher than the required design (up to a 100-year post-development inflow) without structural failure or causing downstream harm or safety risks.

Any stormwater management facility that has a paved roadway as the lower berm, and therefore cannot provide a traditional berm emergency spillway, is not required to provide 0.5 feet of freeboard above the elevation required to safely pass the 100-year post-development inflow, but is required to show that the design is stable and no significant undermining, scour or erosion will occur.

b. Emergency spillways shall be constructed of reinforced concrete, vegetated earth, or riprap in accordance with generally accepted engineering practices. All emergency spillways shall be constructed so that the detention basin berm is protected against erosion. The minimum capacity of all emergency spillways shall be the peak flow rate from the 100-year design storm. The dimensions of the emergency spillways can be determined from the Centre County Erosion and Sediment Control Handbook. Emergency spillways shall extend along the upstream and downstream berm embankment slopes. Protection should be provided on the upstream embankment a minimum of three (3) feet below the spillway crest elevation. The downstream slope of the spillway shall, as a minimum, extend to the toe of the berm embankment. The emergency spillway shall not be located on or discharge over uncompacted earthen fill and/or easily erodible material.

c. Rock-filled gabions may be used where combination berm and emergency spillway structures are required to prevent concentrated flows. The Municipal Engineer may require the use of open concrete lattice blocks, stone riprap, or concrete spillways when slopes would exceed four (4) feet horizontal to one (1) foot vertical and spillway velocities might exceed Soil Conservation Service standards for the particular soils involved.

5. Antiseep Collars: Antiseep collars shall be installed around the principal pipe barrel within the normal saturation zone of the detention basin berms. The antiseep collars and their connections to the pipe barrel shall be watertight. The antiseep collars shall extend a minimum of two (2) feet beyond the outside of the principal pipe barrel. The maximum spacing between collars shall be fourteen (14) times the minimum projection of the collar measured perpendicular to the pipe.

6. Slope of Detention Basin Embankment: The top or toe of any slope shall be located a minimum of ten (10) feet from any property line. Whenever possible the side slopes and basin shape shall be amenable to the natural topography. Straight side slopes and rectangular basins shall be avoided whenever possible.

a. Exterior slopes of compacted soil shall not exceed three (3) feet horizontal to one (1) foot vertical, and may be further reduced if the soil has unstable characteristics.

b. Interior slopes of the basin shall not exceed two (2) feet horizontal to one (1) foot vertical, except with approval of the Municipality.

7. Width of Berm: The minimum top width of detention basin berms shall be ten (10) feet.

8. Slope of Basin Bottom: In order to ensure proper drainage of the detention basin, a minimum grade of two (2) percent shall be maintained for all basins used exclusively for peak runoff control. Water quality or recharge basins with filtration systems incorporated into them may have a minimum grade of 1/2 (0.5) percent.

9. The lowest floor elevation of any structure constructed adjacent to a detention basin or other stormwater facility shall be two (2) feet above the detention basin berm. The distance between any structure and any stormwater facility shall be a minimum of 25 feet.

10. Landscaping and planting specifications must be provided for all stormwater management basins and be specific for each type of basin.

11. Basins should be lined with impervious liners only in areas with a high risk of sinkhole formation or potential groundwater contamination as determined by a geotechnical engineer. However, where a liner is deemed necessary or appropriate, the use of controlled, compacted natural clay liners, for SWM basins should be considered. Locally available clay, when properly installed, can provide near impervious conditions (approximately E-6 cm/s or less). Some of the advantages of using controlled, compacted, natural clay soil liners are:

    a. Can offer better long-term solution as a basin liner versus geosynthetics because of greater thickness and the ability to withstand settlement;

    b. Can be constructed to allow relatively uniform leakage rates to facilitate ground-water recharge but not to an excessive degree that overloads karst bedrock;

    c. When properly constructed in two or more 8- to 10-inch thick lifts, rapid movement of surface water through the clay liner is eliminated (rapid leaks can occur in geosynthetic lined basins due to poor seaming, punctures, or other factors);

    d. Cleaning/maintenance of clay-lined stormwater basins will be easier/safer versus geosynthetic liners which could easily be damaged during maintenance operations; and

    e. The abundance of clayey soils (derived from limestone residuum) within the Spring Creek Watershed can provide adequate, cost effective, soil resources for construction of clay liner systems at most development projects.

    However, the installation of any low permeability clay liner system needs to be carefully controlled and the designer needs to ensure that specifications meet standards to ensure integrity.

C. Construction of Basins

1. Basins shall be installed prior to or concurrent with any earthmoving or land disturbances, which they will serve. The phasing of their construction shall be noted in the narrative and on the plan. Basins that include water quality or recharge components, shall have those components installed in such a manner as to not disturb or diminish their effectiveness.

2. Construction specifications in accordance with the minimum criteria of the municipality must be provided for all embankments pursuant to Section 1b of this Ordinance.

3. Compaction test reports shall be kept on file at the site and be subject to review at all times with copies being forwarded to the Municipal Engineer upon request.

4. When rock is encountered during the excavation of a pond, it shall be removed to an elevation of at least twelve (12) inches below the proposed basin floor (for a manufactured liner, 24 to 30 inches). All exposed cracks and fissures are to be structurally filled.

5. Temporary and permanent grasses or stabilization measures shall be established on the sides and base of all earthen basins within 15 days of construction.

D. Construction Inspection

Inspections may be conducted by the Municipal Engineer during the construction of the stormwater management basin and facilities. Such inspections do not constitute approval of construction methods or materials.

E. Special Use Basins

1. The design and construction of multiple use stormwater detention facilities are strongly encouraged. In addition to stormwater management, facilities should, where appropriate, allow for recreational uses including: ball fields, play areas, picnic grounds, etc. Provision for parking facilities within basins and permanent wet ponds with stormwater management capabilities may also be appropriate. Prior approval and consultation with the Municipality are required before design. Multiple use basins should be constructed so that potentially dangerous conditions are not created.

   Water quality basins or recharge basins that are designed for a slow release of water or other extended detention ponds are not permitted for recreational uses, unless the ponded areas are clearly separated and secure.

2. Multiple Development Basins: Stormwater management facilities designed to serve more than one property or development in the same watershed are encouraged. Staged construction of existing or proposed multiple-use detention facilities by several developers in conjunction with watershed development is encouraged. Each developer shall be responsible for the incremental increase in runoff generated by the respective development and incremental construction

improvements necessary for the overall detention facility. Prior approval and consultation with the Municipality is required before design of such facilities.

3. Alternative Detention Facilities: Alternative stormwater detention facilities including rooftop, subsurface basins or tanks and in-pipe detention storage, or other approved alternative designs are permitted as determined by the Municipal Engineer.

Section 308. Easements

Stormwater management facilities located outside of existing or proposed right-of-ways shall be located within and accessible by easements as follows:

A. Drainage Easements: Where a tract is traversed by a watercourse, drainage-way, channel or stream, there shall be provided a drainage easement paralleling the line of such watercourse, drainage-way, channel or stream. The width of the drainage easement will be adequate to preserve the unimpeded flow of natural drainage in the 100-year flood plain, in accordance with computed top widths for water surface elevations determined under Section 1.1. of this article.

B. Access Easements: Where proposed stormwater management facilities are not adjacent to proposed or existing public right-of-ways or are not accessible due to physical constraints, as determined by the Municipal Engineer, a twenty (20) foot wide passable access easement specifying rights of entry shall be provided. Access easements shall provide for vehicle ingress and egress on grades of less than ten (10) percent for carrying out inspection or maintenance activities.

C. Maintenance Easements: A maintenance easement shall be provided which encompasses the stormwater facility and appurtenances and provides for access for maintenance purposes. The maintenance easement must be located outside of 100-year surface elevation and the stormwater facility and appurtenances.

D. Easements shall stipulate that no trees, shrubs, structures, excavation or fill be placed and no regrading be performed within the area of the easement without written approval from the Municipality upon review by the Municipal Engineer. Upon approval of the Municipal Engineer, such landscaping may be placed in maintenance easements, provided it does not impede access.

E. Whenever practicable, easements shall be parallel with and conjunctive to property lines of the subdivision.

F. All easement agreements shall be recorded with a reference to the recorded easement indicated on the site plan. The format and content of the easement agreement shall be reviewed and approved by the Municipal Engineer and Solicitor.

G. When stormwater conveyance pipes or channels are located in undedicated land, they shall be placed within a drainage easement not less than twenty (20) feet wide as approved by the Municipal Engineer.

ARTICLE IV – DRAINAGE PLAN REQUIREMENTS

Section 401. General Requirements

From and after the date of enactment of this Ordinance, a stormwater management plan and other information specified herein, shall be submitted to the Municipality for all lands subdivided or for which land development plans are prepared after the enactment of this Ordinance. A stormwater management plan and other information specified herein shall be submitted at the same time and together with submission of a preliminary subdivision or land development plan, along with a completed checklist supplied by the Municipality indicating the items contained within the submission.

Such plans and information shall be considered part of said zoning and subdivision documents and shall be reviewed in accordance with procedures established thereunder. Preliminary approval or final approval of a subdivision or land development plan, or the issuance of a zoning permit, shall be contingent upon submission of a stormwater management plan and other materials specified herein, and approval of the stormwater management plan in accordance with provisions of this Ordinance.

All stormwater management plans shall be submitted to the Municipal Engineer for review and comment. Such review shall include a statement by the Municipal Engineer specifying the provisions of this Ordinance, which have not been met by the plan as submitted.

Once a stormwater management plan has been approved together with a subdivision or land development plan approval, or together with the issuance of a zoning permit, said stormwater management plan shall be valid only for the subdivision, land development, or zoning permit approved. Any further development on the lot or lots requiring a revision of the approved plan or other construction or activities as defined by Municipal Zoning Regulations shall require the submission of a new, amended, or revised stormwater management plan and other information specified herein.

Section 402. Drainage Plan Contents

The Drainage Plan shall consist of all applicable calculations, maps, and plans. A note on the maps shall refer to the associated computations and erosion and sediment pollution control plan by title and date. The cover sheet of the computations and erosion and sediment pollution control plan shall refer to the associated maps by title and date. All Drainage Plan materials shall be submitted to the municipality in a format that is clear, concise, legible, neat, and well organized; otherwise, the Drainage Plan shall be disapproved and returned to the Applicant.

Said plan shall be prepared by a registered professional land surveyor, qualified geologist, landscape architect, architect, or engineer licensed in the State of Pennsylvania, with said preparer's seal and registration number affixed to the plan.

The following items shall be included in the Drainage Plan:

A. Stormwater Management Report

1. General description of project.

2. General description of permanent stormwater management techniques, including construction specifications and materials to be used for stormwater management facilities.

3. Complete hydrologic, hydraulic, and structural computations for all stormwater management facilities.

4. A written maintenance plan for all stormwater features including detention facilities and other stormwater management elements.

5. Identification of ownership and maintenance responsibility for all permanent stormwater management facilities.

6. The stormwater management report must include a narrative which clearly discusses the project and summary tables which, at a minimum, provides the following information:

   a. Narrative

      - The overall stormwater management concept
      - The expected project schedule
      - Location map
      - Total site area – pre and post, which must be equal or have an explanation as to why it is not
      - Total site impervious area
      - Total off-site areas
      - Number of stormwater management facilities (ponds), if applicable
      - Type of development
      - Pre-development land use
      - Whether site is underlain by carbonate geology
      - Whether site is a water quality sensitive (WQS) development
      - Whether site is in a defined sensitive area
      - Types of water quality and recharge systems used, if applicable
      - Other pertinent information, as required

   b. Summary Tables

      - Pre-development

        - Hydrologic soil group (HSG) assumptions, curve numbers (CN)
        - Computation of average slope, hydraulic length, computed time of concentration
        - Required peak rate of runoff

      - Post-development

- Undetained areas, areas to ponds
- Land use for each subarea
- Hydrologic soil group (HSG) assumptions, curve numbers (CN)
- Time of concentration computed for each subarea
- Post-development peak rate of runoff routed to ponds and out
- Pond maximum return period design data including: maximum water surface elevation, berm elevation, and emergency spillway elevation
- Water quality depth and volume requirements
- Recharge volume requirements
- Morphology requirements
- Capture volumes required

Reports that do not clearly indicate the above information may be rejected for review by the Municipality's Engineer or representative and will be returned to the applicant.

B. Plans for tracts of less than 20 acres shall be drawn at a scale of one inch equals no more than 50 feet; for tracts of 20 acres or more, plans shall be drawn at a scale of one inch equals no more than 100 feet. Plans shall be submitted on the following sheet sizes: 18' x 24", 24" x 36", or 36" x 42". All lettering shall be drawn to a size to be legible if the plans are reduced to half size. All sheets comprising a submission shall be on one size.

The following information, unless specifically exempted in writing by the Municipal Engineer, must be shown on the plans, prepared in a form which meets the requirements for recording in the Office of the Recorder of Deeds of Centre County, Pennsylvania. The contents of the map(s) shall include, but not be limited to:

1. The name of the development, the name and address of the owner of the property, and the name of the individual or firm preparing the plan.

2. The date of submission and revision.

3. The location of the project relative to highways, municipalities or other identifiable landmarks.

4. Existing contours at intervals of two feet. In areas of steep slopes (greater than 15 percent), five-foot contour intervals may be used.

5. Existing streams, lakes, ponds, or other bodies of water within the project area.

6. Other physical features including flood hazard boundaries, sinkholes, closed depressions, wetlands, streams, existing drainage courses, areas of natural vegetation to be preserved, and the total extent of the upstream area draining through the site. In Addition, any areas necessary to determine downstream impacts, where required for proposed stormwater management facilities must be shown.

7. The locations of all existing and proposed utilities, sanitary sewers, and water lines within 20 feet of property lines.

8. An overlay showing soil names and boundaries, including rock outcrops.

9. Total area of impervious surfaces proposed.

10. Proposed structures, roads, paved areas, and buildings.

11. Final contours at intervals of two feet. In areas of steep slopes (greater than 15 percent), five-foot contour intervals may be used.

12. A graphic and written scale.

13. A North arrow.

14. The total tract boundary and size with distances marked to the nearest foot and bearings to the nearest degree.

15. Existing and proposed land use(s).

16. A key map showing all existing man-made features beyond the property boundary that would be affected by the project and the extent of the watershed or sub-area that drains through the project site.

17. Horizontal and vertical profiles of all open channels, including hydraulic capacity.

18. Overland drainage paths.

19. Access easements around all stormwater management facilities that would provide ingress to and egress from a public right-of-way.

20. A note on the plan indicating the location and responsibility for maintenance of stormwater management facilities that would be located off-site. All off-site facilities shall meet the performance standards and design criteria specified in this Ordinance.

21. A construction detail of any improvements made to sinkholes and the location of all notes to be posted, as specified in this Ordinance.

22. Complete drainage systems for the site, including details for construction. All existing drainage features, which are to be incorporated in the design, shall be so identified. If the site is to be developed in stages, a general drainage plan for the entire site shall be presented with the first stage and appropriate development stages for the drainage system shall be indicated.

23. Location and selected plan material used for vegetative filter paths to sinkholes, and the location of all notices to be posted.

24. A statement, signed by the landowner, acknowledging the stormwater management system to be a permanent fixture that can be altered or removed only after approval of a revised plan by the municipality.

25. A note indicating that As-Built drawings will be provided by the Developer for all stormwater facilities prior to occupancy, or the release of the surety bond.

26. The following signature block for the registered professional preparing the Stormwater Management Plan:

    "I, _____, hereby certify that the Stormwater Management Plan meets all design standards and criteria of the Spring Creek Act 167 Stormwater Management Ordinance."

27. The following signature block for the Township Engineer reviewing the Stormwater Management Plan:

    "I, _____, have reviewed this Stormwater Management Plan in accordance with the Design Standards and Criteria of the Spring Creek Act 167 Stormwater Management Ordinance."

28. The location of all erosion and sedimentation control facilities.

C. Supplemental Information

1. A soil erosion and sediment pollution control plan, where applicable, including all reviews and approvals, as required by PA DEP.

2. Soils investigation report, including boring logs, compaction requirements, and recommendations for construction of detention basins.

3. Karst Features Identification and Analysis Reports and a hydrogeologic assessment of the effects of runoff on sinkholes.

4. The effect of the project (in terms of runoff volumes and peak flows) on adjacent properties and on any existing municipal stormwater collection system that may receive runoff from the project site.

5. A Declaration of Adequacy and Highway Occupancy Permit from the PA DOT District Office when utilization of a PA DOT storm drainage system is proposed.

6. All permits required by the Pennsylvania Department of Environmental Resources, and Army Corps of Engineers and other regulatory agencies.

D. Stormwater Management Facilities

1. All stormwater management facilities must be located on a plan and described in detail.

2. When groundwater recharge methods such as seepage pits, beds or trenches are used, the locations of existing and proposed septic tank infiltration areas and wells must be shown.

3. All calculations, assumptions, and criteria used in the design of the stormwater management facilities must be shown.

   a. A sketch of the berm embankment and outlet structure indicating the embankment top elevation, embankment side slopes, top width of embankment, emergency spillway elevation, perforated riser dimensions, pipe barrel dimensions and dimensions and spacing of antiseep collars.

   b. Design computations for the pipe barrel and riser.

   c. A plot or table of the stage-storage (acre-feet vs. elevation) and all supporting computations.

   d. Flood routing computations.

   e. A detailed plan of the trash rack and anti-vortex device.

4. Record Set (As-Built) Plans: At the completion of the project, and as a prerequisite for the release of the guarantee or issuance of an occupancy permit, the owner or his representative shall:

   a. Provide certification of completion from a registered professional verifying that all permanent facilities have been constructed according to the plans and specifications and approved revisions thereto; and

   b. Provide a set of approved stormwater management plan drawings showing all approved revisions and elevations and inverts to all manholes, inlets, pipes, and stormwater control facilities.

Section 403. Plan Submission

For all activities regulated by this Ordinance, the steps below shall be followed for submission. For any activities that require a PA DEP Joint Permit Application and regulated under Chapter 105 (Dam Safety and Waterway Management) or Chapter 106 (Floodplain Management) of PA DEP's Rules and Regulations, require a PA DOT Highway Occupancy Permit, or require any other permit under applicable state or federal regulations, the permit(s) shall be part of the plan.

A. The Drainage Plan shall be submitted by the Developer as part of the Preliminary Plan submission for the Regulated Activity.

B. Four (4) copies of the Drainage Plan and Stormwater Management Report shall be submitted.

C. Distribution of the Drainage Plan will be as follows:

1. Two (2) copies to the Municipality accompanied by the requisite Municipal Review Fee, as specified in this Ordinance.

2. One (1) copy to the Municipal Engineers.

3. One (1) copy to the County Planning Commission/Department.

Section 404. Drainage Plan Review

A. The Municipal Engineer shall review the Drainage Plan for consistency with the adopted Spring Creek Watershed Act 167 Stormwater Management Plan. The Municipality shall require receipt of a complete plan, as specified in this Ordinance.

B. The Municipal Engineer shall review the Drainage Plan for any submission or land development against the municipal subdivision and land development ordinance provisions not superseded by this Ordinance.

C. For activities regulated by this Ordinance, the Municipal Engineer shall notify the Municipality in writing, within ___ calendar days, whether the Drainage Plan is consistent with the Stormwater Management Plan. Should the Drainage Plan be determined to be consistent with the Stormwater Management Plan, the Municipal Engineer will forward an approval letter to the Developer with a copy to the Municipal Secretary.

D. Should the Drainage Plan be determined to be inconsistent with the Stormwater Management Plan, the Municipal Engineer will forward a disapproval letter to the Developer with a copy to the Municipal Secretary citing the reason(s) for the disapproval. Any disapproved Drainage Plans may be revised by the Developer and resubmitted consistent with this Ordinance.

E. For Regulated Activities specified in Sections 104.C and 104.D of this Ordinance, the Municipal Engineer shall notify the Municipal Building Permit Officer in writing, within a time frame consistent with the Municipal Building Code and/or Municipal Subdivision Ordinance, whether the Drainage Plan is consistent with the Stormwater Management Plan and forward a copy of the approval/disapproval letter to the Developer. Any disapproved drainage plan may be revised by the Developer and resubmitted consistent with this Ordinance.

F. For Regulated Activities requiring a PA DEP Joint Permit Application, the Municipal Engineer shall notify PA DEP whether the Drainage Plan is consistent with the Stormwater Management Plan and forward a copy of the review letter to the Municipality and the Developer. PA DEP may consider the Municipal Engineer's review comments in determining whether to issue a permit.

G. The Municipality shall not approve any subdivision or land development for Regulated Activities specified in Sections 104.A and 104.B of this Ordinance if the Drainage Plan has been found to be inconsistent with the Stormwater Management Plan, as determined by the Municipal Engineer. All required permits from PA DEP must be obtained prior to approval.

H. The Municipal Building Permit Office shall not issue a building permit for any Regulated Activity specified in Section 104 of this Ordinance if the Drainage Plan has been found to be inconsistent with the Stormwater Management Plan, as determined by the Municipal Engineer, or without considering the comments of the Municipal Engineer. All required permits from PA DEP must be obtained prior to issuance of a building permit.

I. The Developer shall be responsible for completing an "As-Built Survey" of all stormwater management facilities included in the approved Drainage Plan. The As-Built Survey and an explanation of any discrepancies with the design plans shall be submitted to the Municipal Engineer for final approval. In no case shall the Municipality approve the As-Built Survey until the Municipality receives a copy of an approved Declaration of Adequacy, Highway Occupancy Permit from the PA DOT District Office, and any applicable permits from PA DEP.

J. The Municipality's approval of a Drainage Plan shall be valid for a period not to exceed _____ ( ) years. This ____-year time period shall commence on the date that the Municipality signs the approved Drainage Plan. If stormwater management facilities included in the approved Drainage plan have not been constructed, or if an As-Built Survey of these facilities has not been approved within this _____-year time period, then the Municipality may consider the Drainage plan disapproved and may revoke any and all permits. Drainage Plans that are considered disapproved by the Municipality shall be resubmitted in accordance with Section 407 of this Ordinance.

Section 405. Modification of Plans

A modification to a submitted Drainage Plan for a development site that involves a change in stormwater management facilities or techniques, or that involves the relocation or re-design of stormwater management facilities, or that is necessary because soil or other conditions are not as stated on the Drainage Plan as determined by the Municipal Engineer, shall require a resubmission of the modified Drainage Plan consistent with Section 404 of this Ordinance and be subject to review as specified in Section 405 of this Ordinance.

A modification to an already approved or disapproved Drainage Plan shall be submitted to the Municipality, accompanied by the applicable review. A modification to a Drainage Plan for which a formal action has not been taken by the Municipality shall be submitted to the Municipality, accompanied by the applicable Municipality Review Fee.

Section 406. Resubmission of Disapproved Drainage Plans

A disapproved Drainage Plan may be resubmitted, with the revisions addressing the Municipal Engineer's concerns documented in writing, to the Municipal Engineer in accordance with Section 404 of this Ordinance and be subject to review as specified in Section 405 of this Ordinance. The applicable Municipality Review Fee must accompany a resubmission of a disapproved Drainage Plan.

ARTICLE V – INSPECTIONS

Section 501. Schedule of Inspections

A. The Municipal Engineer or his municipal assignee shall inspect all phases of the installation of the permanent stormwater management facilities.

B. During any stage of the work, if the Municipal Engineer determines that the permanent stormwater management facilities are not being installed in accordance with the approved Stormwater Management Plan, the Municipality shall revoke any existing permits until a revised Drainage Plan is submitted and approved, as specified in this Ordinance.

ARTICLE VI – FEES AND EXPENSES

Section 601. General

The fees required by this Ordinance are the Municipal Review Fee and the Municipality Review Fee. The Municipal Review fee shall be established by the Municipality to defray review costs incurred by the Municipality and the Municipal Engineer. All fees shall be paid by the Applicant.

Section 602. Municipality Drainage Plan Review Fee

The Municipality shall establish a Review Fee Schedule by resolution of the municipal governing body based on the size of the Regulated Activity and based on the Municipality's costs for reviewing Drainage Plans. The Municipality shall periodically update the Review Fee Schedule to ensure that review costs are adequately reimbursed.

Section 603. Expenses Covered by Fees

The fees required by this Ordinance shall at a minimum cover:

A. Administrative Costs.

B. The review of the Drainage Plan by the Municipality and the Municipal Engineer.

C. The site inspections.

D. The inspection of stormwater management facilities and drainage improvements during construction.

E. The final inspection upon completion of the stormwater management facilities and drainage improvements presented in the Drainage Plan.

F. Any additional work required to enforce any permit provisions regulated by this Ordinance, correct violations, and assure proper completion of stipulated remedial actions.

ARTICLE VII – MAINTENANCE RESPONSIBILITIES

Section 701. Stormwater Management Controls and Facilities

Stormwater management controls and facilities as defined here include all structural and non-structural stormwater conveyance and management controls including water quantity and quality Best Management Practices.

Section 702. Performance Guarantee

The applicant should provide a financial guarantee to the Municipality for the timely installation and proper construction of all stormwater management controls as required by the approved stormwater plan and this ordinance equal to the full construction cost of the required controls.

Section 703. Maintenance Responsibilities

A. The Drainage Plan for the development site shall contain an operation and maintenance plan prepared by the developer and approved by the Municipal Engineer. The operation and maintenance plan shall outline required routine maintenance actions and schedules necessary to ensure proper operation and function of the facility(ies).

B. The responsible party or entity responsible for the maintenance must also be identified. The Drainage Plan for the development site shall establish responsibilities for the continuing operation and maintenance of all proposed stormwater control facilities and temporary permanent erosion control facilities, consistent with the following principals:

1. If a development consists of structures or lots that are to be separately owned and in which streets, sewers and other public improvements are to be dedicated to the municipality, stormwater control facilities may also be dedicated to and maintained by the municipality.

2. If a development site is to be maintained in a single ownership or if sewers and other public improvements are to be privately owned and maintained, then the ownership and maintenance of stormwater control facilities shall be the responsibility of the owner or private management entity.

   Facilities may be incorporated within individual lots so that the respective lot owners will own and be responsible for maintenance in accordance with recorded deed restriction. A description of the facility or system and the terms of the required maintenance shall be incorporated as part of the deed to the property.

   Ownership and maintenance may be the responsibility of a Property Owners Association. The stated responsibilities of the Property Owners Association in terms of owning and maintaining the stormwater management facilities shall be submitted with final plans for determination of their adequacy, and upon their approval shall be recorded with the approved subdivision plan among the deed records of Centre County, Pennsylvania. In addition, the approved subdivision plan and any deed written from said plan for a lot or lots shown herein shall

contain a condition that it shall be mandatory for the owner or owners of said lot to be members of said Property Owners Association.

For stormwater management facilities that are proposed as part of the site development plan, the developer will be required to execute a developer agreement and a maintenance agreement with the Municipality for the construction and continued maintenance of the facilities prior to the signature approval on the final plan. Access for inspection by the municipality of all such facilities deemed critical to the public welfare at any reasonable time shall be provided.

C. The governing body, upon recommendation of the Municipal Engineer, shall make the final determination on the continuing maintenance responsibilities prior to final approval of the stormwater management plan. The governing body reserves the right to accept the ownership and operating responsibility for any or all of the stormwater management controls.

Section 704. Maintenance Agreement for Privately Owned Stormwater Facilities

A. Prior to final approval of the site's stormwater management plan, the property owner shall sign and record a maintenance agreement covering all stormwater control facilities that are to be privately owned.

B. Other items may be included in the agreement where determined necessary to guarantee the satisfactory maintenance of all facilities. The maintenance agreement shall be subject to the review and approval of the municipal solicitor and governing body.

Section 705. Municipal Stormwater Maintenance Fund

A. If stormwater facilities are accepted by the municipality for dedication, persons installing stormwater facilities shall be required to pay a specified amount to the Municipal Stormwater Maintenance Fund to help defray costs of periodic inspections and maintenance expenses. The amount of the deposit shall be determined as follows:

1. If the stormwater facility is to be owned and maintained by the municipality, the deposit shall cover the estimated costs for maintenance and inspections for ten (10) years. The Municipal Engineer will establish the estimated costs utilizing information submitted by the applicant.

2. The amount of the deposit to the fund shall be converted to present worth of the annual series values. The Municipal Engineer shall determine the present worth equivalents, which shall be subject to the approval of the municipal governing body.

B. If a stormwater facility is proposed that also serves as a recreation facility (e.g., ballfield, lake), the municipality may reduce or waive the amount of the maintenance fund deposit based upon the value of the land for public recreation purpose.

C. If at some future time a stormwater facility (whether publicly or privately owned) is eliminated due to the installation of storm sewers or other facilities, the unused portion of the maintenance fund deposit will be applied to the cost of abandoning the facility and connecting to the storm sewer system or other facility. Any amount of the deposit remaining after the costs of abandonment are paid will be returned to the depositor.

Section 706. Post-Construction Maintenance Inspections

A. Stormwater facilities should be inspected by the land owner/developer or responsible entity (including the Municipal Engineer for dedicated facilities) on the following basis:

1. Annually

2. During or immediately after every ten-year or greater storm event.

B. The entity conducting the inspection should be required to submit a report to the municipality regarding the condition of the facility and recommending necessary repairs, if needed.

C. Maintenance inspections may be performed by the Municipality to ensure proper functioning of all stormwater facilities. These inspections may, at a minimum, be performed annually and/or following major storm events.

If the Municipality determines at any time that any permanent stormwater facility has been eliminated, altered or improperly maintained, the owner of the property shall be advised of corrective measures required and given three (3) days to initiate appropriate action in accordance with a time schedule dictated by the Municipality. If such action is not taken by the property owner, the Municipality may cause the work to be done and backcharge all costs to the property owners.

ARTICLE VIII – ENFORCEMENT AND PENALTIES

Section 801. Right-of-Entry

Upon presentation of proper credentials, duly authorized representatives of the municipality may enter, at reasonable times, upon any property within the municipality to inspect the condition of the stormwater structures and facilities in regard to any aspect regulated by this Ordinance.

Section 802. Notification

In the event that a person fails to comply with the requirements of this Ordinance, or fails to conform to the requirements of any permit issued hereunder, the municipality shall provide written notification of the violation. The notice will direct the responsible party to comply with all the terms of this Ordinance within seven (7) days, or such additional period, not to exceed thirty (30) days, as the designated Municipal representative shall deem reasonable. In addition, the designated Municipal representative shall give notice to the owner, applicant, developer, property manager or other person responsible for the property or the violation that if the violation is not corrected, the municipality may correct the same and charge the landowner or other person responsible the cost thereof plus penalties as specified herein for failure to comply.

Such notice may be delivered by the United States mail, first class, postage prepaid, or by certified or registered mail; or by personal service; or if the property is occupied, by posting the notice at a conspicuous place upon the affected property.

Such notification shall set forth the nature of the violation(s) and establish a time limit for correction of these violations(s). Failure to comply within the time specified shall subject such person to the penalty provision of this Ordinance. All such penalties shall be deemed cumulative and resort by the municipality from pursuing any and all other remedies. It shall be the responsibility of the owner of the real property on which any Regulated Activity is proposed to occur, is occurring, or has occurred, to comply with the terms and conditions of this Ordinance.

Section 803. Enforcement

The municipal governing body is hereby authorized and directed to enforce all of the provisions of this Ordinance. All inspections regarding compliance with the drainage plan shall be the responsibility of the Municipal Engineer or other qualified persons designated by the municipality.

A. A set of design plans approved by the municipality shall be on file at the site throughout the duration of the construction activity. Periodic inspections may be made by the municipality or designee during construction.

B. Adherence to Approved Plan

It shall be unlawful for any person, firm or corporation to undertake any regulated activity under Section 104 on any property except as provided for in the approved drainage plan and pursuant to the requirements of this Ordinance. It shall be unlawful to alter or

remove any control structure required by the drainage plan pursuant to this Ordinance or to allow the property to remain in a condition which does not conform to the approved drainage plan.

C. At the completion of the project, and as a prerequisite for the release of the performance guarantee, the owner or his representatives shall:

1. Provide a certification of completion from an engineer, architect, surveyor or other qualified person verifying that all permanent facilities have been constructed according to the plans and specifications and approved revisions thereto.

2. Provide a set of as-built drawings.

D. After receipt of the certification by the municipality, a final inspection shall be conducted by the governing body or its designee to certify compliance with this Ordinance.

E. Prior to revocation or suspension of a permit, the governing body will schedule a hearing to discuss the non-compliance if there is no immediate danger to life, public health or property.

F. Suspension and Revocation of Permits

1. Any permit issued under this ordinance may be suspended or revoked by the governing body for:

a. Non-compliance with or failure to implement any provision of the permit.

b. A violation of any provision of this ordinance or any other applicable law, ordinance, rule or regulation relating to the project.

c. The creation of any condition or the commission of any act during construction or development which constitutes or creates a hazard or nuisance, pollution or which endangers the life or property of others, or as outlined in Article IX of this Ordinance.

2. A suspended permit shall be reinstated by the governing body when:

a. The Municipal Engineer or his designee has inspected and approved the corrections to the stormwater management and erosion and sediment pollution control measure(s), or the elimination of the hazard or nuisance;

b. The governing body is satisfied that the violation of the ordinance, law, or rule and regulation has been corrected.

A permit that has been revoked by the governing body cannot be reinstated. The applicant may apply for a new permit under the procedures outlined in this Ordinance.

G. Occupancy Permit

An occupancy permit shall not be issued unless the certification of compliance pursuant to Section 902.D has been secured. The occupancy permit shall be required for each lot owner and/or developer for all subdivisions and land development in the municipality.

Section 804. Public Nuisance

A. The violation of any provision of this Ordinance is hereby deemed a Public Nuisance.

B. Each day that a violation continues shall constitute a separate violation.

Section 805. Penalties

A. Anyone violating the provisions of this Ordinance shall be guilty of a misdemeanor, and upon conviction shall be subject to a fine of not more than $_____ for each violation and recoverable with costs. Each day that the violation continues shall be a separate offense.

B. In addition, the municipality, through its solicitor, may institute injunctive, mandamus or any other appropriate action or proceeding at law or in equity for the enforcement of this Ordinance. Any court of competent jurisdiction shall have the right to issue restraining orders, temporary or permanent injunctions, mandamus or other appropriate forms of remedy or relief.

C. The cost of removal, fine and penalties hereinabove mentioned may be entered by the Municipality as a lien against such property, or properties of individual members of a Property Owners Association, in accordance with the existing provisions of the law.

Section 806. Appeals

A. Any person aggrieved by any action of the Municipality or its designee, relevant to the provisions of this Ordinance, may appeal to the Municipal Zoning Hearing Board within thirty (30) days of that action.

B. Any person aggrieved by any decision of the municipal Zoning Hearing Board, relevant to the provisions of this Ordinance, may appeal to the County Court of Common Pleas in the county where the activity has taken place within thirty (30) days of the Zoning Hearing Board's decision.

ENACTED and ORDAINED at a regular meeting of the [Name of the municipal governing body] on the _____ day of _____, 19__. This Ordinance shall take effect immediately.

_____
[Name],[Title]

_____
[Name],[Title]

_____
[Name],[Title]

ATTEST:

_____
[name], Secretary (type or print)

I hereby certify that the foregoing Ordinance was advertised in the [name of newspaper] on [date], a newspaper of general circulation in the municipality and was duly enacted and approved as set forth at a regular meeting of the [name of municipal governing body] held on [date].

_____
[name], Secretary

APPENDIX A

STORMWATER MANAGEMENT DESIGN CHARTS AND TABLES

TABLE A-1
IDF REGION 2DESIGN STORM RAINFALL

TABLE A-2
RUNOFF CURVE NUMBERS
(FROM NRCS (SCS) TR-55)

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RATIONAL RUNOFF COEFFICIENTS
(ARON CURVES)

TABLE A-4
RATIONAL RUNOFF COEFFICIENTS
(RAWLS, WONG, McCUEN)

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MANNING ROUGHNESS COEFFICIENTS
FOR OPEN CHANNELS AND MANNING N VALUES FOR SHEET FLOW

TABLE A-6
MANNING ROUGHNESS COEFFICIENTS
FOR PIPES

TABLE A-7
PERMISSIBLE VELOCITIES FOR CHANNELS

TABLE A-8
SOILS IDENTIFIED IN THE CENTRE COUNTY SOIL SURVEY
AS ON FLOOD PLAINS OR ON TERRACES ABOVE FLOOD PLAINS

TABLE A-1
IDF REGION 2 DESIGN STORM RAINFALL

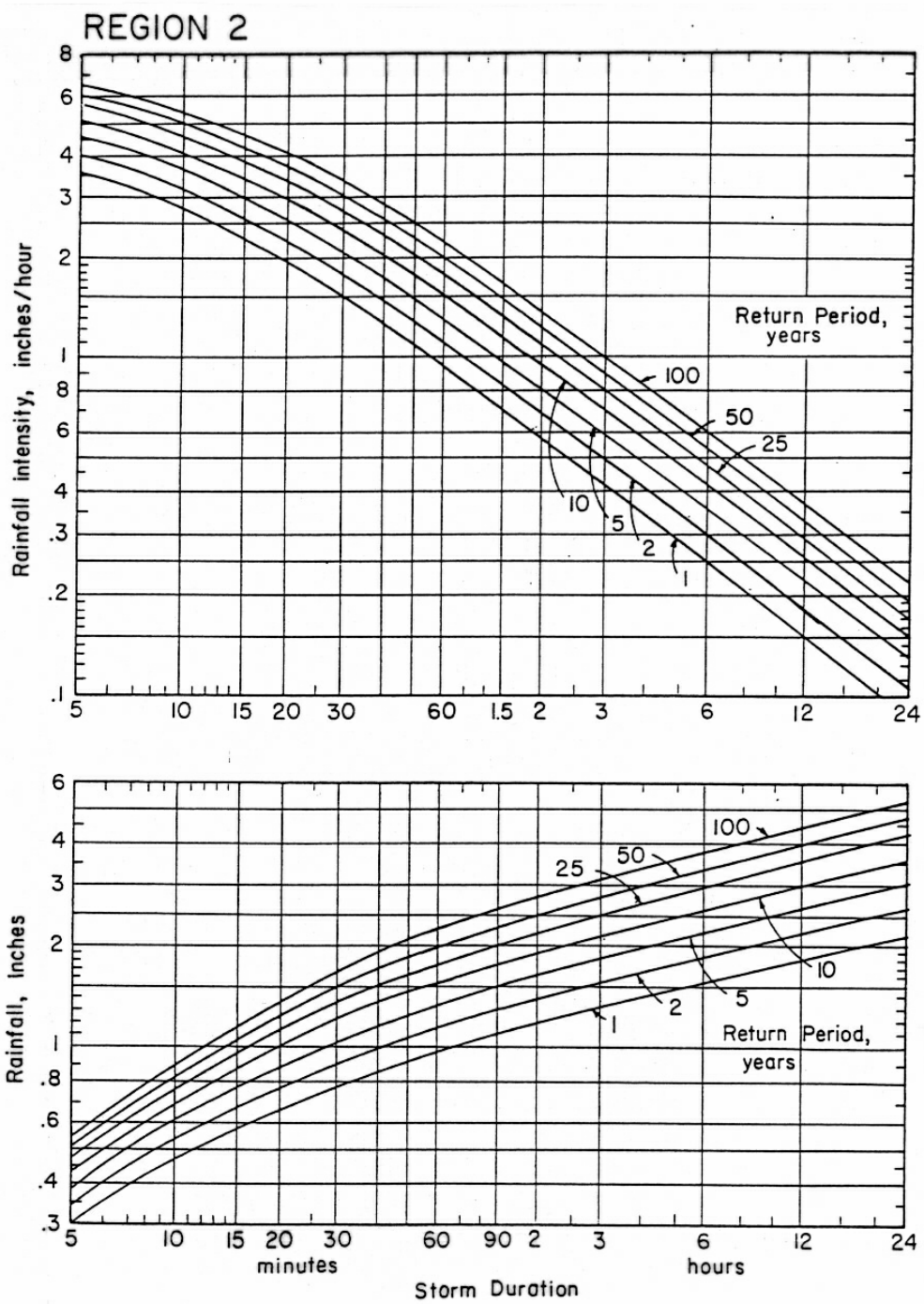

TABLE A-2
RUNOFF CURVE NUMBERS
(FROM NRCS (SCS) TR-55)

TR-55

Cover Description Land/Use Cover Type	Average Imperviousness (percent)	Curve Numbers For Hydrologic Soil Group A	B	C	D
Open space (lawns, parks, golf courses, cemeteries, etc.):					
Good condition (grass cover greater than 75%	n/a[a]	39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)	n/a	98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)	n/a	98	98	98	98
Paved; open ditches (including right-of-way)	n/a	98	98	98	98
Gravel (including right-of-way)		76	85	89	91
Urban Districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential Districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Woods:	n/a	30	55	71	77
Brush:		35	56	70	77
Meadow: (In good condition)		30	58	71	78

[a] Not applicable

Source: United States Department of Agriculture, Soil Conversation Service, Engineering Division, 1986, "Urban Hydrology for Small Watersheds," Technical Release 55, Washington, DC.

TABLE A-3
RATIONAL RUNOFF COEFFICIENTS
(ARON CURVES)

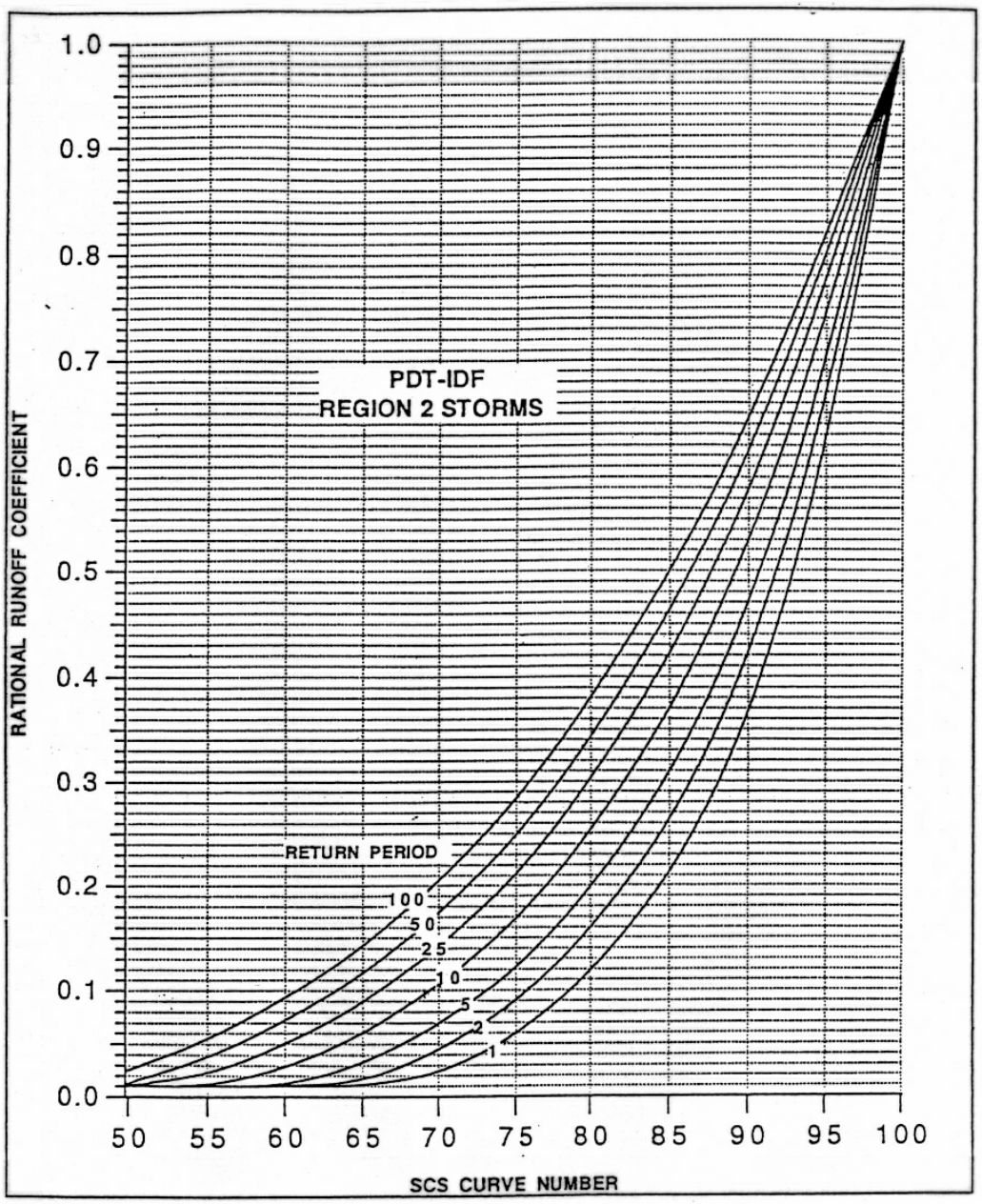

TABLE A-4
RATIONAL RUNOFF COEFFICIENTS
(RAWLS, WONG, McCUEN)

Runoff Coefficients for the Rational Formula
By Hydrologic Soil Group and Overland Slope (%)

	A			B			C			D		
Land Use	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+
Cultivated Land	0.08[a]	0.13	0.16	0.11	0.15	0.21	0.14	0.19	0.26	0.18	0.23	0.31
	0.14[b]	0.18	0.22	0.16	0.21	0.28	0.20	0.25	0.34	0.24	0.29	0.41
Pasture	0.12	0.20	0.30	0.18	0.28	0.37	0.24	0.34	0.44	0.30	0.40	0.50
	0.15	0.25	0.37	0.23	0.34	0.45	0.30	0.42	0.52	0.37	0.50	0.62
Meadow	0.10	0.16	0.25	0.14	0.22	0.30	0.20	0.28	0.36	0.24	0.30	0.40
	0.14	0.22	0.30	0.20	0.28	0.37	0.26	0.35	0.44	0.30	0.40	0.50
Forest	0.05	0.08	0.11	0.08	0.11	0.14	0.10	0.13	0.16	0.12	0.16	0.20
	0.08	0.11	0.14	0.10	0.14	0.18	0.12	0.16	0.20	0.15	0.20	0.25
Residential	0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
Lot Size 1/8 Acre	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Lot Size 1/4 Acre	0.22	0.26	0.29	0.24	0.29	0.33	0.27	0.31	0.36	0.30	0.34	0.40
	0.30	0.34	0.37	0.33	0.37	0.42	0.36	0.40	0.47	0.38	0.42	0.52
Lot Size 1/3 Acre	0.19	0.23	0.26	0.22	0.26	0.30	0.25	0.29	0.34	0.28	0.32	0.39
	0.28	0.32	0.35	0.30	0.35	0.39	0.33	0.38	0.45	0.36	0.40	0.50
Lot Size 1/2 Acre	0.16	0.20	0.24	0.19	0.23	0.28	0.22	0.27	0.32	0.26	0.30	0.37
	0.25	0.29	0.32	0.28	0.32	0.36	0.31	0.35	0.42	0.34	0.38	0.48
Lot Size 1 Acre	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.29	0.35
	0.22	0.26	0.29	0.24	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46
Industrial	0.67	0.68	0.68	0.68	0.68	0.69	0.68	0.69	0.69	0.69	0.69	0.70
	0.85	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Commercial	0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
	0.88	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
Streets	0.70	0.71	0.72	0.71	0.72	0.74	0.72	0.73	0.76	0.73	0.75	0.78
	0.76	0.77	0.79	0.80	0.82	0.84	0.84	0.85	0.89	0.89	0.91	0.95
Open Space	0.05	0.10	0.14	0.08	0.13	0.19	0.12	0.17	0.24	0.16	0.21	0.28
	0.11	0.16	0.20	0.14	0.19	0.26	0.18	0.23	0.32	0.22	0.27	0.39
Parking	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87
	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97

[a] Runoff coefficients for storm recurrence intervals less than 25 years.
[b] Runoff coefficients for storm recurrence intervals of 25 years or more.

Source: Rawls, W.J., S.L. Wong and R.H. McCuen, 1981, "Comparison of Urban Flood Frequency Procedures," Preliminary Draft, U.S. Department of Agriculture, Soil Conservation Service, Beltsville, MD.

TABLE A-5
MANNING ROUGHNESS COEFFICIENTS
FOR OPEN CHANNELS AND MANNING N VALUES FOR SHEET FLOW

	Manning's n range
I. Closed conduits:	
A. Concrete pipe	0.011-0.013
B. Corrugated-metal pipe or pipe arch:	
1. 2 1/2 by 1/2 inch corrugation (riveted) pipe:	
a. Plain or fully coated	0.024
b. Paved invert (range values are for 25 and 50 percent of circumference paved):	
(1) Flow full depth	0.021-0.018
(2) Flow 0.8 depth	0.021-0.016
(3) Flow 0.6 depth	0.019-0.013
2. 6 by 2-inch corrugation (field bolted)	0.030
C. Cast-iron pipe, uncoated	0.013
D. Steel pipe	0.009-0.011
E. Monolithic concrete:	
1. Wood forms, rough	0.015-0.017
2. Wood forms, smooth	0.012-0.014
3. Steel forms	0.012-0.013
F. Cemented rubble masonry walls:	
1. Concrete floor and top	0.017-0.022
2. Natural floor	0.019-0.025
II. Open channels, lined (straight alignment):	
A. Concrete, with surfaces as indicated:	
1. Formed, no finish	0.013-0.017
2. Trowel finish	0.012-0.014
3. Float finish	0.013-0.015
4. Float finish, some gravel on bottom	0.015-0.017
5. Gunite, good section	0.016-0.019
6. Gunite, wavy section	0.018-0.022
B. Concrete, bottom float finished, sides as indicated:	
1. Dressed stone in mortar	0.015-0.017
2. Random stone in mortar	0.017-0.020
3. Cement rubble masonry	0.020-0.025
4. Cement rubble masonry, plastered	0.016-0.020
5. Dry rubble (riprap)	0.020-0.030
C. Gravel bottom, sides as indicated:	
1. Formed concrete	0.017-0.020
2. Random stone in mortar	0.020-0.023
3. Dry rubble (riprap)	0.023-0.033
D. Asphalt	
1. Smooth	0.013
2. Rough	0.016
E. Concrete-lined excavated rock:	
1. Good section	0.017-0.020
2. Irregular section	0.022-0.027

	Manning's n range
III. Open channels, excavated (straight alignment, natural lining):	
A. Earth, uniform section:	
1. Clean, recently completed	0.016-0.018
2. Clean, after weathering	0.018-0.020
3. With short grass, few weeds	0.022-0.027
4. In gravely soil, uniform section, clean	0.022-0.025
B. Earth, fairly uniform section:	
1. No vegetation	0.022-0.025
2. Grass, some weeds	0.025-0.030
3. Dense weeds or aquatic plants in deep channels	0.030-0.035
4. Sides clean, gravel bottom	0.025-0.030
5. Sides clean, cobble bottom	0.030-0.040
C. Dragline excavated or dredged:	
1. No vegetation	0.028-0.033
2. Light brush on banks	0.035-0.050
D. Rock:	
1. Based on design section	0.035
2. Based on actual mean section:	
a. Smooth and uniform	0.035-0.040
b. Jagged and irregular	0.040-0.045
E. Channels not maintained, weeds and brush uncut:	
1. Dense weeks, high as flow depth	0.080-0.120
2. Clean bottom, brush on sides	0.050-0.080
3. Clean bottom, brush on sides, highest stage of flow	0.070-0.110
4. Dense brush, high stage	0.100-0.140
IV. Channels and swales w/maintained Vegetation (values shown are for velocities of 2 & 6 f.p.s.):	
A. Depth of flow up to 0.7 foot:	
1. Bermudagrass, Kentucky bluegrass, buffalograss	
a. Mowed to 2 inches	0.045-0.070
b. Length 4-6 inches	0.050-0.090
2. Good stand, any grass:	
a. Length about 12 inches	0.090-0.180
b. Length about 24 inches	0.150-0.300
3. Fair stand, any grass:	
a. Length about 12 inches	0.080-0.140
b. Length about 24 inches	0.130-0.250
B. Depth of flow 0.7-1.5 feet:	
1. Bermudagrass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.035-0.050
b. Length 4 to 6 inches	0.040-0.060
2. Good stand, any grass:	
a. Length about 12 inches	0.070-0.120
b. Length about 24 inches	0.100-0.200
3. Fair stand, any grass:	
a. Length about 12 inches	0.060-0.100
b. Length about 24 inches	0.090-0.170

	Manning's n range
V. Street and expressway gutters:	
A. Concrete gutter, troweled finish	0.012
B. Asphalt pavement:	
1. Smooth texture	0.013
2. Rough texture	0.016
C. Concrete gutter with asphalt pavement	
1. Smooth	0.013
2. Rough	0.015
D. Concrete pavement:	
1. Float finish	0.014
2. Broom finish	0.015
E. For gutters with small slope, where sediment may accumulate, increase above values of x by	0.002
VI. Natural stream channels:	
A. Minor streams (surface width at flood stage less than 100 feet):	
1. Fairly regular section:	
a. Some grass & weeds, little or no brush	0.030-0.035
b. Dense growth of weeks, depth of flow materially greater than weed height	0.035-0.050
c. Some weeks, light brush on banks	0.035-0.050
d. Some weeks, heavy brush on banks	0.050-0.070
e. Some weeks, dense willows on banks	0.060-0.080
f. For trees within channel with branches submerged at high stage, increase all above values by	0.010-0.020
2. Irregular sections, with pools, slight channel meander; increase given in 1 a-e about	0.010-0.020
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage	
a. Bottom of gravel, cobbles and few boulders	0.040-0.050
b. Bottom of cobbles, with large boulders	0.050-0.070
B. Floodplains (adjacent to natural streams):	
1. Pasture, no brush:	
a. Short grass	0.030-0.035
b. High grass	0.035-0.050
2. Cultivated areas:	
a. No crop	0.030-0.040
b. Mature row crops	0.035-0.045
c. Mature field crops	0.040-0.050
3. Heavy weeds, scattered brush	0.050-0.070
4. Light brush and trees:	
a. Winter	0.050-0.060
b. Summer	0.060-0.080
5. Medium to dense brush:	
a. Winter	0.070-0.110
b. Summer	0.100-0.160
6. Dense willows, summer, not bent over by current	0.150-0.200
7. Cleared land w/tree stumps, 100-150 per acre:	
a. No sprouts	0.040-0.050
b. With heavy growth of sprouts	0.060-0.080
8. Heavy stand of timber, a few down trees, little undergrowth:	
a. Flood depth below branches	0.100-0.120
b. Flood depth reaches branches	0.120-0.160
C. Major streams (surface width at flood stage more than 100 feet): Roughness coefficient is usually less than for minor streams of less effective resistance offered by irregular banks or vegetation on banks. Values of n may be somewhat reduced. Follows recommendation in publication cited if possible. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of	0.028-0.033

MANNING'S ROUGHNESS COEFFICIENTS FOR SHEET FLOW

SURFACE DESCRIPTION	n[1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover 20%	0.06
Residue cover 20%	0.17

SURFACE DESCRIPTION	n[1]
Grass:	
Short grass prairie	0.15
Dense grasses	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:	
Light underbrush	0.40
Dense underbrush	0.80

Source: Chow, V.T., 1959, "Open Channel Hydraulics," McGraw Hill, New York.

TABLE A-6
MANNING ROUGHNESS COEFFICIENTS
FOR PIPES

Description	"n"
Smooth-roll plastic pipe	0.011
Concrete pipe	0.012
Smooth-lined corrugated metal pipe	0.012
Corrugated plastic pipe	0.024
Annular corrugated steel and aluminum Alloy pipe (plain or polymer coated) 2 2/3" x 1/2' corrugations 3" x 1" corrugations 5" x 1" corrugations 6" x 2" corrugations	 0.024 0.027 0.025 0.033
Helically corrugated steel and aluminum Alloy pipe (plain or polymer coated) 3" x 1", 5" x 1" or 6" x 2" corrugations	0.024
Helically corrugated steel and aluminum Alloy pipe (plain or polymer coated) 2 2/3" x 1/2" corrugations a. lower coefficients● 18" diameter 24" diameter 36" diameter 48" diameter 60" diameter or larger b. Higher coefficients▲	 0.014 0.016 0.019 0.020 0.021 0.024
Annular or Hhlically corrugated steel or aluminum alloy pipe arches or other on-circular conduit (plain or polymer coated)	0.024
Vitrified clay pipe	0.012
Ductile iron pipe	0.013

●Use the lower coefficient if any one of the following conditions apply:

a. A storm pipe longer than 20 diameters, which directly or indirectly connects to an inlet or manhole, located in swales adjacent to shoulders in cut areas, shoulders in cut areas or depressed medians.

b. A storm pipe which is specially designed to perform under pressure.

▲Use the higher coefficient if any one of the following conditions apply:

a. A storm pipe which directly or indirectly connects to an inlet or manhole located in highway pavement sections or adjacent to curb or concrete median barrier.

b. A storm pipe which is shorter than 20 diameters long.

c. A storm pipe which is partly lined helically corrugated metal pipe.

In considering each factor more critical, judgement is necessary if it is kept in mind that any condition that causes turbulence and retards flow results in a greater value of "n."

Outlet velocity for bituminous paved invert shall be determined based on a 25% reduction in Manning's roughness coefficient, "n."

Source: Pennsylvania Department of Transportation Design Manual, Part 2, January 1990.

TABLE A-7
PERMISSIBLE VELOCITIES FOR CHANNELS

Maximum Permissible Velocities in Bare Earth Channels -For Straight Channels where slope < .02 ft/ft

Soil Materials	n*	Clear Water (V fps)	Water Transporting Colloidal Silts (V fps)
Fine sand, noncolloidal	.020	1.50	2.50
Sandy loam, noncolloidal	.020	1.75	2.50
Silt loam, noncolloidal	.020	2.00	3.00
Alluvial silts, noncolloidal	.020	2.00	3.50
Ordinary firm loam	.020	2.50	3.50
Stiff clay, very colloidal	.025	3.75	5.00
Alluvial silts, colloidal	.025	3.75	5.00
Shales and hardpan	.025	6.00	6.00
Fine Gravel	.020	2.50	5.00
Graded loam - cobbles (when noncolloidal)	.030	3.75	5.00
Graded silt - cobbles (when noncolloidal)	.030	4.00	5.50
Coarse gravel noncolloidal	.025	4.00	6.00
Cobbles and shingles	.035	5.00	5.50

* Listed n values assume good to excellent construction techniques which produce uniform channel dimensions. Values should be adjusted, by use of SCS Engineering Handbook #5, Supplement B, for other construction conditions.

TABLE A-5.2 Maximum Permissible Velocities for Channels Lines with Vegetation

Cover	Slope Range Percent	Permissible Velocity ft/sec. Erosion[1] Resistant Soil	Easily[2] Eroded Soil
Kentucky Bluegrass Tall Fescue	< 5 5-10 > 10	7[3] 6[3] 5	5 4 3
Grass Mixture Reed Canarygrass	< 5 5-10	5 4	4 3
Sericea Lespedeza Weeping Lovegrass Redtop Red Fescue	< 5	3.5	2.5
Annuals temporary cover only Sudangrass	< 5	3.5	2.5

[1] Cohesive (clayey) fine grain soils and coarse grain soils with a plasticity index of 10 to 40 (CL, CH, SC, & GC).
[2] Soils that do not meet the requirements for erosion resistant soils.
[3] Use velocities exceeding 5 ft./sec. only where good cover and proper maintenance can be obtained.

TABLE A-8
SOILS IDENTIFIED IN THE CENTRE COUNTY SOIL SURVEY
AS ON FLOOD PLAINS OR ON TERRACES ABOVE FLOOD PLAINS

Allegheny Series	Allegheny silt loam (AlB)
Atkins Series	Atkins silt loam (At)
Basher Series	Basher loam (Ba)
Chagrin Series	Chagrin Soils (Ch)
Dunning Series	Dunning silty clay loam (Du)
Lindside Series	Lindside soils (Lx)
Melvin Series	Melvin silt loam (Mm)
Monongahela Series	Monongahela silt loam (MoB)
Philo Series	Philo loam (Ph), Philo and Atkins very stony soils (Pk)
Pope Series	Pope soils (Po)
Purdy Series	Purdy silt loam (Pu)
Tyler Series	Tyler silt loam (Ty)

APPENDIX B

WATERSHED MAPS

Sensitive Land Areas for Well Head Protection Data Source

Well Fields 1 and 3: Harter and Thomas Well Fields

Municipality: Harris, Ferguson, and College Townships
Well Owner: State College Water Authority
Includes wells: H7, H8, H11, H14, H22, H25
Protection Area: One-year zone of contribution
Source: Nittany Geoscience, February 1992, Figure 4

Well Field 5

Municipality: Ferguson Township
Well Owner: State College Water Authority
Includes wells: F55, F57
Protection Area: One-year zone of contribution
Source: Nittany Geoscience, February 1992, Figure 4

Well Field 6

Municipality: Benner and Patton Townships
Well Owner: State College Water Authority
Includes wells: B62, B63, B64, B65
Protection Area: One-year zone of contribution + direct upslope lands
Source: Nittany Geoscience, February 1992, Figure 4

PSU Golf Course Well Field

Municipality: Ferguson Township and the Borough of State College
Well Owner: Penn State University
Includes wells: PS28A, PS 37
Protection Area: One-year zone of contribution
Source: Nittany Geoscience, January, Figure 5

PSU Big Hollow Well Field

Municipality: Patton, Ferguson, and College Townships
Well Owner: Penn State University
Includes wells: PS2, PS14, PS16, PS17, PS24, PS26
Protection Area: One-year zone of contribution
Source: Nittany Geoscience, January, Figure 5

PSU Houserville Well Field

Municipality: Ferguson Township
Well Owner: Penn State University
Includes wells: PS33, PS 34, PS35
Protection Area: One-year zone of contribution
Source: Nittany Geoscience, January, Figure 5

Existing Well and Spring

Municipality: Ferguson Township
Well Owner: State College Water Authority
Includes wells: F3
Protection Area: 400' Radius + direct upslope lands

Ridgemont Wells

Municipality: Patton Township
Well Owner: Ridgemont Water Authority
Includes wells: P1, P2
Protection Area: 400' Radius

Spring Creek Park, Lemont #4, Lemont #5,and Rogers Wells, and Bathgate Springs

Municipality: College Township
Well Owner: College Township Water Authority
Includes wells: C1, C2, C3, C4, C5
Protection Area: 400' Radius

DEFINITIONS

For the purposes of this ACT 167 Plan, certain terms and words used herein shall be interpreted as follows:

A. Words used in the present tense include the future tense; the singular number includes the plural, and the plural number includes the singular; words of masculine gender include feminine gender; and words of feminine gender include masculine gender.

B. The word "includes" or "including" shall not limit the term to the specific example but is intended to extend its meaning to all other instances of like kind and character.

C. The word "person" includes an individual, firm, association, organization, partnership, trust, company, corporation, or any other similar entity.

D. The words "shall" and "must" are mandatory; the words "may" and "should" are permissive.

E. The words "used or occupied" include the words "intended, designed, maintained, or arranged to be used, occupied or maintained".

Abutment – The part of a structure (arch or bridge) that receives thrust or pressure. Usually implies the fill portion of a bond or basin earthen section for stormwater management purposes.

Aggradation – The process of temporary filling in of a channel, or a raising of the grade, due to changes in a channel geometry or discharge. Generally accompanied by a widening of the channel.

Accelerated Erosion – The removal of the surface of the land through the combined action of man's activity and the natural processes of a rate greater than would occur because of the natural process alone.

Agricultural Activities – The work of producing crops and raising livestock including tillage, plowing, disking, harrowing, pasturing and installation of conservation measures. Construction of new buildings or impervious area is not considered an agricultural activity.

Alluvial Soils – Those areas delineated pursuant to the Centre County, Pennsylvania, Soil Survey, August 1981, and subsequent revisions.

Alteration – As applied to land, a change in topography as a result of the moving of soil and rock from one location or position to another; also the changing of surface conditions by causing the surface to be more or less impervious; land disturbance.

Antecedent Moisture – Soil-moisture content preceding a given storm.

Antecedent Runoff Condition (ARC) – An NRCS defined index (formerly AMC) of watershed wetness that relates the volume of runoff as a function of precipitation that occurred prior to the event. Usually based on a period of five (5) days.

Armoring – The process of a channel bed becoming protected by the stabilization of the bed materials up to some limiting discharge or event.

Applicant – A landowner or developer who has filed an application for approval to engage in any Regulated Activities as defined in Section 104 of this Ordinance.

Bankfull Discharge – The discharge related to depth of flow at the top of bank prior to spilling over into the flood plain. Usually used as the channel-forming discharge for downstream changes in channel geometry.

Baseflow – Stream discharge derived from effluent groundwater seepage. A time-based definition relating to sustained or fair-weather runoff largely composed of groundwater effluent. Outflow from extensive groundwater aquifers, which are recharged by water percolating down through the soil mantle to the water table.

Bed – Stream channel bottom.

Bed Forms – Flow induced bed features that form in alluvial channels and directly affect the roughness or flow resistance.

BMP (Best Management Practice) – Stormwater structures, facilities and techniques to maintain or improve the water quality of surface runoff.

Buffer Area – Area that is protected from development in order to prevent degradation of the water body or water quality.

Cartway – Drivable road surface.

Capture Depth/Volume – Depth or volume of runoff captured from a given area and either allowed to evaporate, infiltrate, or be discharged through a subsurface drain connected to a pond principal spillway at a negligible rate.

Carbonate – A sediment formed by the organic or inorganic precipitation of mineral compounds characterized by the fundamental chemical ion CO3, the principal element in limestone and dolomite strata.

Channel – A perceptible natural or artificial waterway, which periodically or continuously contains moving water having a definite bed and banks, which confine the water.

Channel Erosion – The widening, deepening, and headward cutting of small channels and waterways, due to erosion caused by moderate to large floods.

Cistern – An underground reservoir or tank for storing rainwater.

Closed Or Undrained Depression – In a Karst geologic area a distinct bowl-shaped depression in the land surface; size and amplitude are variable; drainage is internal. It differs from a sinkhole in that the ground surface is unbroken and usually occurs in greater density per unit area.

Cluster Development – Low impact method of land development consisting of clusters of houses and impervious areas surrounded by pervious open space. In general reduces the overall impervious area of the site; however, without considering the actual hydrologic processes that occur at a site can still result in adverse stormwater runoff.

CFS – Cubic feet per second – standard measurement of flow.

Conservation District – The Centre County Conservation District.

Credits – A deduction from the required amount. In this Ordinance, implies reduction of required water quality volumes due to using a recommended practice.

Culvert – A structure with appurtenant works which carries a stream under or through an embankment or fill.

Curve Number (CN) – a parameter defined by the NRCS to represent the transformation of rainfall to runoff.

Dam – An artificial barrier, together with its appurtenant works, constructed for the purpose of impounding or storing water or another fluid or semifluid, or a refuse bank, fill or structure for highway, railroad or other purposes which does or may impound water or another fluid or semifluid.

Dbh – Diameter at Breast Height - the diameter of a tree at a height of four and one-half (4-1/2) feet above the ground, on the uphill side.

Degradation – The process of wearing down by erosion due to changes in a channel geometry or discharge.

Design Storm – The magnitude and temporal distribution of precipitation from a storm event measured in probability of occurrence (e.g., a 5-year storm) and duration (e.g., 24 hours), used in the design and evaluation of stormwater management systems.

Designee – The agent of a Planning Commission and/or agent of the governing body involved with the administration, review or enforcement of any provisions of this Ordinance by contract or memorandum of understanding.

Detention Basin – An impoundment structure designed to manage stormwater runoff by temporarily storing the runoff and releasing it at a predetermined rate.

Developer – A person, partnership, association, corporation, or other entity, or any responsible person therein or agent thereof, that undertakes any Regulated Activity of this Ordinance.

Development Site – The specific tract of land for which a Regulated Activity is proposed.

Direct Runoff – A time-based definition of runoff entering stream channels promptly after rainfall or snow melt; superimposed on baseflow, it forms the bulk of the flood hydrograph. The sum of channel precipitation, overland flow and subsurface stormflow.

Discharge – A rate of flow or flow from a hydraulic structure.

Diversion – A channel, berm, or dike constructed upslope of a project for the purpose of diverting stormwater away from the unprotected slope.

Dolomite – (1) A mineral consisting of calcium magnesium carbonate found as compact lime stone; or (2) limestone or marble rich in magnesium carbonate.

Dominant Discharge – Generally considered the channel forming discharge.

Downslope Property Line – That portion of the property line of the lot, tract, or parcels of land being developed located such that all overland or pipe flow from the site would be directed towards it.

Drainage Conveyance Facility – A Stormwater Management Facility designed to transmit stormwater runoff and shall include streams, channels, swales, pipes, conduits, culverts, storm sewers, etc.

Drainage Easement – A right granted by a landowner to a grantee, allowing the use of private land for stormwater management purposes.

Drainage Permit – A permit issued by the Township governing body after the drainage plan has been approved. Said permit is issued prior to or with the final Township approval.

Drainage Plan – The documentation of the stormwater management system, if any, to be used for a given development site, the contents of which are established in Section 403.

Drainage Way – The natural or man-made path of surface water from a given area.

Earth Disturbance – Any activity including, but not limited to, construction, mining, timber harvesting and grubbing which alters, disturbs, and exposes the existing land surface.

Effective Rainfall (effective precipitation) – That part of rainfall (or precipitation) that produces runoff. Rainfall minus evaporation and interception storage.

Emergency Spillway – A part of a stormwater detention facility designed for safe passage of flows higher than the facility was designed for. Generally based on the 100-year runoff event for stormwater management ponds.

Erosion – The movement of soil particles by the action of water, wind, ice, or other natural forces.

Erosion and Sediment Pollution Control Plan – A plan that is designed to minimize accelerated erosion and sedimentation.

Evaporation – The process by which moisture passes into the atmosphere as vapor.

Evaportranspiration – Water withdrawn from a land area by evaporation from water surfaces and moist soil, and by plant transpiration.

Existing Conditions – The initial condition of a project site prior to the proposed construction. If the initial condition of the site is undeveloped land, the land use shall be considered as "meadow" unless the natural land cover is proven to generate lower curve numbers or Rational "C" value, such as forested lands.

Exfiltration – The process by which water or moisture moves from a subsurface trench, bed, or other feature into the subsoil. Exfiltration is best measured by a soil's percolation rate.

Flood – A general but temporary condition of partial or complete inundation of normally dry land areas from the overflow of streams, rivers, and other waters of the Commonwealth.

Floodplain – Any land area susceptible to inundation by water from any natural source or delineated by applicable Department of Housing and Urban Development, Federal Insurance Administration Flood Hazard Boundary - Mapped as being a special flood hazard area. Also included are areas that comprise Group 13 Soils, as listed in Appendix A of the Pennsylvania Department of Environmental Protection (PA DEP) Technical Manual for Sewage Enforcement Officers (as amended or replaced from time to time by PA DEP).

Floodway – The channel of the watercourse and those portions of the adjoining floodplains that are reasonably required to carry and discharge the 100-year frequency flood. Unless otherwise specified, the boundary of the floodway is as indicated on maps and flood insurance studies provided by FEMA. In an area where no FEMA maps or studies have defined the boundary of the 100-year frequency floodway, it is assumed - absent evidence to the contrary - that the floodway extends from the stream to 50 feet from the top of the bank of the stream.

Forest Management/Timber Operations – Planning and activities necessary for the management of forestland. These include timber inventory and preparation of forest management plans, silvicultural treatment, cutting budgets, logging road design and construction, timber harvesting, site preparation and reforestation.

Freeboard – A vertical distance between the elevation of the design high-water and the top of a dam, levee, tank, basin, or diversion ridge. The space is required as a safety margin in a pond or basin.

Gabion – A large rectangular box of heavy gage wire mesh, which holds large cobbles and boulders. Used in streams and ponds to change flow patterns, stabilize banks, or prevent erosion.

Geologic Formation – The basic or fundamental rock stratigraphic unit in the local classification of rocks, consisting of a body of rock (usually a sedimentary stratum or strata but also igneous or metamorphic) generally characterized by some degree of internal lithologic homogeneity or distinctive lithologic features (such as chemical composition, structures, textures, gross aspect of fossils or time of deposition.) Typically used for mapping the geology of an area.

Geologic Member – A rock stratigraphic unit which is subordinate (a subject) of a formation. This unit is not necessarily mapable and is usually a unified subdivision of local extent that may or may not be contained in more than one formation.

Ghost Lake – A body of standing water occurring in a sinkhole or closed depression of a Karst region that is usually visible after sufficient precipitation has occurred. They may form from slow permeability of soils, raises in the water table or the development of a natural liner of slow permeable clays or soils.

Grade – A slope, usually of a road, channel or natural ground specified in percent and shown on plans as specified herein. (To) Grade - to finish the surface of a roadbed, top of embankment or bottom of excavation.

Grassed Waterway – A natural or constructed waterway, usually broad and shallow, covered with erosion-resistant grasses, used to conduct surface water from cropland.

Groundwater Flow – Flow of water underground. For the purposes of this Ordinance will loosely refer to all flows other than surface runoff.

Groundwater Recharge – Replenishment of existing natural underground water supplies. Traditional land development activities reduce groundwater recharge.

HEC-1 – Model developed by COE Hydrologic Engineering Center to simulate the surface runoff response of a river basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components.

HEC-HMS – Windows version of HEC-1.

Hydraulic Routing – A method of modeling the movements of water based on both the momentum and continuity equations.

Hydrologic Cycle – The sequence of conditions through which water passes from vapor in the atmosphere through precipitation upon land or water surfaces and ultimately back to the atmosphere through evaporation.

Hydrologic Processes – For the intent of this Ordinance they are processes that control the hydrologic cycle in relation to runoff consisting of: diffuse or conduit flows interflow components, infiltration excess, saturation excess, exfiltration, etc.

Hydrologic Routing – The procedure used to derive a downstream hydrograph from an upstream hydrograph, or tributary hydrograph, and from considerations of local inflow by solving the storage equation.

Hydrologic Soil Group (HSG) – An NRCS soil designation based on infiltration capacity used in determining hydrologic soil-cover complexes, which are used in estimating runoff from rainfall.

IDF – Intensity, duration, frequency.

Impervious Surface – A surface that prevents the percolation of water into the ground.

Impoundment – A retention or detention basin designed to retain stormwater runoff and release it at a controlled rate.

Infiltration – The passage of water through the surface of the soil, via pores or small openings, into the soil mass.

Infiltration Excess – The concept that surface runoff results from the rate of rainfall exceeding the potential infiltration rate of the soil at some time.

Infiltration Rate – The infiltration rate of a soil is related to the soil's final infiltration capacity and represents the rate at which water enters the soil/air interface at the top of the soil profile. Infiltration rates are measured in units of length / time.

Infiltration Structures – A structure designed to direct runoff into the ground (e.g., french drains, seepage pits, seepage trench).

Initial Abstraction (IA) – Consists mainly of the interception, infiltration, and surface storage of precipitation that occurs priors to the start of runoff.

Inlet – A surface connection to a closed drain. A structure at the diversion end of a conduit. The upstream end of any structure through which water may flow.

Interceptor – A channel, berm, or dike constructed across a slope for the purpose of intercepting stormwater, reducing the velocity of flow, and diverting it to outlets where it may be disposed.

Interflow – An intermediate component of runoff, between overland flow and groundwater flow, made up of subsurface flow which returns to form surface runoff without reaching the water table before arriving at the watershed outlet.

Karst – A type of topography that is formed over limestone, dolomite, or gypsum by bedrock solution, and that is characterized by closed depressions or sinkholes, caves, and underground drainage (from AGI, Glossary of Geology, 1972).

Lag – In relation to the runoff hydrograph, the time from the center of mass of the excess rainfall to some defined point in the runoff hydrograph. For NRCS methods this point is defined as the time to the peak.

Lag Equation – The original NRCS timing parameter developed to couple unit hydrograph theory and curve number based runoff volumes.

Land Development – (i) The improvement of one lot or two or more contiguous lots, tracts, or parcels of land for any purpose involving (a) a group of two or more buildings, or (b) the division or allocation of land or space between or among two or more existing or prospective occupants by means of, or for the purpose of streets, common areas, leaseholds, condominiums, building groups, or other features; (ii) Any subdivision of land; (iii) Development in accordance with Section 503(1.1) of the PA Municipalities Planning Code.

Land/Earth Disturbance – Any activity involving grading, tilling, digging, or filling of ground or stripping of vegetation or any other activity that causes an alteration to the natural condition of the land.

Land Use – The primary application employed in an area.

Level Spreader – A device used to spread out stormwater runoff uniformly over the ground surface as sheet flow (i.e., not through channels). The purpose of level spreaders are to prevent concentrated, erosive flows from occurring, and to enhance infiltration.

Limestone – A rock that, by accumulation of organic remains, consists mainly of calcium carbonate.

Lineaments – Straight or gently curved, lengthy features frequently expressed topographically as depressions or lines on the earth's surface. They can be more easily observed at a height of 100 meters or more and are usually found by researching aerial photographs or satellite photography. They are usually located in areas of faulting or in dense jointing along some rock stratigraphy.

Lined or Liner – The surface condition of a swale or channel, or a subsurface layer within a stormwater pond used to restrict the infiltration of water such as a bentonite or rigid PVC liner.

Low Flow Channel – An incised or paved channel from inlet to outlet in a dry basin which is designed to carry low runoff flows and/or base flow directly to the outlet without detention.

Low Impact Design – Land development designs resulting in minor impacts related to stormwater management.

Main Stem (Main Channel) – Any stream segment or other runoff conveyance facility used as a reach in the Spring Creek hydrologic model.

Manning Equation in (Manning formula) – A method for calculation of velocity of flow (e.g., feet per second) and flow rate (e.g., cubic feet per second) in open channels based upon channel shape, roughness, depth of flow and slope. "Open channels" may include closed conduits so long as the flow is not under pressure.

Minimum Allowable Discharge – In relation to this Stormwater Management Ordinance, the minimum rate that can be discharged for any drainage area for design storm events up to and including the 10-year event regardless of the modeled pre-development runoff estimate.

Modified Puls – Model consisting of repetitive solutions of the continuity equation based on assumptions that (1) water in reservoir is horizontal and (2) outflow is a unique function of storage.

Modified Rational Method – Method of using unit hydrographs in conjunction with the rational equation peak runoff rate to conduct reservoir routings.

Municipality – Any of the several municipalities within the basin consisting of: Bellefonte Borough, Benner Township, Boggs Township, Centre Hall Borough, College Township, Ferguson Township, Halfmoon Township, Harris Township, Milesburg Borough, Patton Township, Potter Township, Spring Township, State College Borough, Walker Township, Pennsylvania.

Municipal Engineer – A professional engineer licensed in the Commonwealth of Pennsylvania and duly appointed by the subject municipality as their representative. In the event that a Stormwater Utility is formed, all references to the Municipal Engineer shall be considered to also imply the Stormwater Utility Engineer.

Nonpoint Source Pollution – Pollution that enters a watery body from diffuse origins in the watershed and does not result from discernible, confined, or discrete conveyances.

NRCS – Natural Resource Conservation Service (previously SCS).

Nuisance Flooding – Flooding that is generally not life threatening, but causes extreme inconveniences and financial burdens on those impacted by the flooding.

Open Channel – A drainage element in which stormwater flows with an open surface. Open channels include, but shall not be limited to, natural and man-made drainageways, swales, streams, ditches, canals, and pipes flowing partly full.

Outfall – Point where water flows from a conduit, stream, or drain.

Outlet – Points of water disposal from a stream, river, lake, tidewater or artificial drain.

PA DEP – Pennsylvania State Department of Environmental Protection.

PA DOT – Pennsylvania State Department of Transportation.

Parking Lot Storage – Involves the use of impervious parking areas as temporary impoundments with controlled release rates during rainstorms.

Partial Source Area (PSA) – The concept that surface runoff is controlled primarily by certain areas of a watershed, generally consisting of areas adjacent to streams or drainage-ways or shallow soils where the soil becomes rapidly saturated. Refer to VSA and saturation excess.

Peak Discharge – The maximum rate of stormwater runoff from a specific storm event.

Percolation Rate – The rate at which water moves through a soil profile. Percolation rates are measured in units of time / length.

Pipe – A culvert, closed conduit, or similar structure (including appurtenances) that conveys stormwater.

Pipe Flow – Concentrated subsurface flow of water in natural conduits or man-made pipes.

Planning Commission – The planning commission of a municipality.

Point Discharge – The discharge from a pipe or channel that concentrates runoff at a single area.

Principal Spillway – A structure used to pass and control discharge from a stormwater management facility.

Probable Maximum Flood (PMF) – The flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in any area. The PMF is derived from the probable maximum precipitation (PMP) as determined on the basis of data obtained from the National Oceanographic and Atmospheric Administration (NOAA).

Probable Maximum Precipitation (PMP) – Refer to PMF (Probable Maximum Flood).

PSU-IV – Runoff model developed by The Pennsylvania State University (PSU), for the use in Pennsylvania for determining runoff peak rates for larger watersheds.

Quickflow – That portion of the total runoff, which is immediately attributable to a precipitation event (see Direct Runoff).

Rational Formula – A rainfall-runoff relation used to estimate peak flow.

Recharge Volume – The volume of water that is required to be recharged from developed sites.

Registered Professional – An individual registered in and licensed by the State of Pennsylvania including, for the purposes of this Ordinance, land surveyors, landscape architects, architects and engineers.

Regulated Activities – Actions or proposed actions that have an impact on stormwater runoff and that are specified in Section 104 of this Ordinance.

Rejecting Sinkhole – A sinkhole that no longer accepts runoff and has started to respond similar to a spring due to changes in the local hydrology.

Release Rate – The percentage of predevelopment peak rate of runoff from a site or subarea to which the post-development peak rate of runoff must be reduced to protect downstream areas.

Riparian Zone – Area adjacent to a natural water course, habitat for special species of flora and fauna.

Retention Basin – An impoundment in which stormwater is stored and not released during the storm event. Stored water may be released from the basin at some time after the end of the storm.

Return Period – The average interval, in years, within which a storm event of a given magnitude can be expected to recur. For example, the 25-year return period rainfall has a 4% probability of occurring in any given year.

Riprap – A combination of large stone, cobbles, and boulders used to line channels, stabilize banks, and reduce runoff velocities.

Riser – A vertical pipe extending from the bottom of a pond that is used to control the discharge rate from the pond for a specified design storm.

Rooftop Detention – Temporary ponding and gradual release of stormwater falling directly onto flat roof surfaces by incorporating controlled-flow roof drains into building designs.

Runoff – Any part of precipitation that flows over the land surface.

Safe Passage – The routing of peak runoff events, usually the 100-year design event, safely through a structure without failure of that structure.

Saturation Excess – The process whereby surface runoff is generated due to saturation of the soil profile.

Scour – Generally refers to the change in a channel configuration provoked by sediment imbalance, due to natural or man made causes, between the supply and transport capacity of the channel.

Sediment Basin – A barrier, dam, retention or detention basin located and designed to retain rock, sand, gravel, silt, or other material transported by water.

Sediment Pollution – The placement, discharge or any other introduction of sediment into the waters of the Commonwealth occurring from the failure to design, construct, implement or maintain control measures and control facilities in accordance with the requirements of this Ordinance.

Sedimentation – The process by which mineral or organic matter is accumulated or deposited by the movement of water.

Seepage Pit/Seepage Trench – An area of excavated earth filled with loose stone or similar coarse material, into which surface water is directed for infiltration into the ground.

Segmental Method – A name sometimes applied to the NRCS's time of concentration method that defines surface runoff components as sheet flow, shallow concentrated flow and channel flow.

Sensitive Water Quality Area – An area protected for its water quality enhancing abilities, or because of its location can cause potentially catastrophic loss or contamination to groundwater reservoirs.

Sheet Flow – Runoff that flows over the ground surface as a thin, even layer, not concentrated in a channel.

Sinkhole – A localized, gradual or rapid sinking of the land surface to a variable depth, occurring in areas of carbonate bedrock; generally characterized by a roughly circular outline, a distant breaking of the ground surface and downward movement of soil into bedrock voids.

Sinkhole Flood Plain – The area inundated by the 100-year, 24-hour storm, assuming no drainage from the sinkhole or closed depression based upon anticipated runoff volumes with maximum development permitted by zoning within the catchment area or area draining to the sinkhole.

Sinuosity – An index used to describe the channel planform. Defined as the ratio of the valley slope to the channel slope, or the ratio of channel length to valley length. Used as an indicator for the age and stability of a channel reach.

Soil-Cover Complex Method – A method of runoff computation developed by the NRCS that is based on relating soil type and land use/cover to a runoff parameter called Curve Number (CN).

Soil Group, Hydrologic – A classification of soils by the Soil Conservation Service into four runoff potential groups. The groups range from A soils, which are very permeable and produce little runoff, to D soils, which are not very permeable and produce much more runoff.

Spillway – A depression in the embankment of a pond or basin which is used to pass peak discharge greater than the maximum design storm controlled by the pond.

Stabilization – The proper placing, grading and/or covering of soil, rock or earth to ensure their resistance to erosion, sliding or other movement.

Storage Indication Method – A reservoir routing procedure based on solution of the continuity equation (inflow minus outflow equals the change in storage) with outflow defined as a function of storage volume and depth.

Storm Frequency – The number of times that a given storm "event" occurs or is exceeded on the average in a stated period of years. See "Return Period".

Storm Sewer – A system of pipes and/or open channels that convey intercepted runoff and stormwater from other sources, but excludes domestic sewage and industrial wastes.

Stormwater Management Facility – Any structure, natural or man-made, that, due to its condition, design, or construction, conveys, stores, or otherwise affects stormwater runoff. Typical stormwater management facilities include, but are not limited to, detention and retention basins, open channels, storm sewers, pipes, and infiltration structures.

Stormwater Management Plan – The plan for managing stormwater runoff in the Spring Creek Watershed adopted by the Centre County Commissioners as required by the Act of October 4, 1978, P.L. 864, (Act 167), and known as the "Spring Creek Watershed Action 167 Stormwater Management Plan.

Stormwater Management Site Plan – The plan prepared by the Developer or his representative indicating how stormwater runoff will be managed at the particular site of interest according to this Ordinance.

Strata – Tabular or sheet-like mass, distinct layers of homogenous or gradational sedimentary material (consolidated rock or unconsolidated earth) of any thickness, visually separable from other layers above and below by a discrete change in the character of the material deposited or by a sharp physical break deposition or both.

Stratigraphic Unit – A stratum or body of strata recognized as a unit in the classification of the rocks of the earth's crust with respect to any specific rock character, property, attribute or for any purpose such as description, mapping, and correlation.

Stream Enclosure – A bridge, culvert or other structure in excess of 100 feet in length upstream to downstream which encloses a regulated water of this Commonwealth.

Structural Fill – For the purposes of this Ordinance, shall imply any soil mass that is compacted in lifts to some tested criteria (standard or modified proctor) such as those under foundations or adjacent to retaining walls. Areas that, for several years after construction, respond to precipitation events similar to impervious areas.

Subarea – The smallest drainage unit of a watershed for which stormwater management criteria have been established in the Stormwater Management Plan.

Subdivision – The division or re-division of a lot, tract, or parcel of land by any means into two or more lots, tracts, parcels or other divisions of land including changes in existing lot lines for the purpose, whether immediate or future, of lease, transfer of ownership, or building or lot development, provided, however, that the subdivision by lease of land for agricultural purposes into parcels of more than ten acres, not involving any new street or easement of access or any residential dwellings, shall be exempt.

Swale – A natural low-lying stretch of land or minor man-made conveyance channel, which gathers or carries surface water runoff.

SWM – Stormwater management.

Timber Operations – See Forest Management.

Time of Concentration (Tc) – Generally considered the time for surface runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed, or in regard to unit hydrographs the time from the end of excess rainfall to the point of inflection on the recession limb of the runoff hydrograph. Refer to Lag Equation and/or Segmental Method.

Topography – The general configuration of a land surface or any part of the earth's surface, including its relief and position of its natural and man-made features. The natural or physical surface features of a region, considered collectively as to its form.

TR-20 – NRCS developed hydrologic model.

TR-55 – A simplified NRCS tabular method of determining runoff developed from TR-20 simulations. Often confused with a true runoff computer model because of its adaptation to computer use.

Undetained Area – An area of a site that cannot be routed to a stormwater management facility because of its location. Generally small areas around access drives or below stormwater management facilities.

Unit Hydrograph – A hydrograph of direct runoff resulting from one (1) inch of effective rainfall generated uniformly over the basin area of a uniform rate over a specified duration.

USDA – United States Department of Agriculture

Vadose Water – Water in the zone of aeration.

Variable Source Area (VSA) – The concept that surface runoff is generated is primarily by small areas within the watershed that expand and contract in response to precipitation events. See PSA and saturation excess.

Velocity – The rate of change in position with respect to time, speed, rate of movements.

Watercourse – A stream of water, river, brook, creek, or a channel or ditch for water, whether natural or manmade.

Water Quality Depth – Depth of precipitation required to be used in computing the water quality volume based on the percentage of imperviousness of a site.

Water Quality Sensitive (WQS) Development – Defined as land development projects that have a high potential to cause catastrophic loss to local water quality and could potentially threaten ground water reservoirs

Water Quality Volume – Volume of runoff required to be controlled from a site in a water quality BMP.

Watershed – The entire region or area drained by a river or other body of water, whether natural or artificial, a drainage basin or sub-basin.

Waters of the Commonwealth – Any and all rivers, streams, creeks, rivulets, ditches, watercourses, storm sewers, lakes, dammed water, wetlands, ponds, springs, and all other bodies or channels of conveyance of surface and underground water, or parts thereof, whether natural or artificial, within or on the boundaries of this Commonwealth.

Water Table – Upper surface of a layer of saturated material in the soil.

Wetland – Those areas that are inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions, including swamps, marshes, bogs, ferns, and similar areas.

APPLICATION OF THE WATER QUALITY, CAPTURE VOLUME, AND RECHARGE DESIGN PROCEEDURES

EXAMPLE 1 – Large Commercial Development Using BMPs

Site Description:

- A 16.67 acre site has a proposed development with 60% impervious area.
- The total proposed site impervious area is 10 acres in size which consists of:
    - The building (1.5 acres total area) roofing material is aluminum panels.
    - All the sidewalks (0.25 acres total area) have cross slopes that allow them to drain to grass buffer strips at least equal in width to the sidewalks.
    - 3 acres of the paved drives and parking areas drain as sheet flow across undisturbed grass buffer areas prior to being conveyed to the stormwater management pond by shallow and wide grass lined swales.
    - 1 acre of the parking lot consists of interlocking concrete paver blocks with grass openings and is used as an overflow parking area.
    - A small closed depression existed on the site that 2 acres of the site drained to. The closed depression was determined to be stable; however, a collection system was designed such that if the closed depression overflowed, the surface runoff would be conveyed to the stormwater management pond. The designer maintained the closed depression and routed 2 acres of parking area to it.
    - The remaining impervious (building and parking) areas (3.75 acres total area) drain directly to the stormwater management pond via storm drain pipes.
- There is a single unlined stormwater management pond that collects almost 100% of the runoff for the proposed impervious area.
- The site is located within a sensitive area.
- The pond discharges 50 feet upslope of the property line to a mild draw located on downstream properties.
- An undisturbed area 50 feet wide was designed into the site plan surrounding the drainage-way for a distance of 500 feet.

Water Quality:

The total site impervious area = 10 acres or 60%; however, the site impervious can be adjusted for the non asphalt roofing material area of 1.5 acres as 10 – 1.5 = 8.5 acres. Therefore the site impervious area (SIA) for water quality = 100 x (8.5)/16.67 = 51%.

Therefore, the $WQ_{depth} = 0.25+(0.012)\ 2.9^{(0.044*51)} = 0.38$ inches

However, since the site is in a sensitive area, the WQ_{depth} = 0.5 inches minimum, therefore using 0.5 inches:

The WQv = is 0.5 inch x 8.5 acres = 4.25 acre/inch = 0.35 acre/feet storage without any credits.

Had the initial computed WQ_{depth} been greater than 0.5 inches, the higher value would have been used.

Because the site uses BMPs, several water quality credits are applicable.

Credit 1 (Drainageway Protection):
If an undisturbed area of 50 feet for 500 feet surrounds the drainage way, a credit for the area (at a 2:1 ratio) is given as:

Credit 1 = 2x(50x500)/43560 = 1.148 acres = 1.148 acres

This is less than 50% of the site water quality impervious area so the full credit is allowed.

Credit 2 (Vegetated Filter Strip/Recharge):
3 acres of pavement and 0.25 acres of sidewalk are diverted across grass buffer areas. The maximum allowable credit for the use of filter strips (NAC buffer areas included) is 50% of the WQ SIA which in this case is 0.5 x 8.5 acres = 4.25 acres.

Credit 2 = 3.25 acres

Total WQ Credits:

Total Credit Area = 1.148 + 3.25 = 4.398 acres

Therefore, the adjusted WQv = 0.5in x (8.5 – 4.398) acres = 2.05 acre-in = 0.17 ac-ft storage.

In addition to the direct credits, two other elements of the site design act as allowable water quality features. These elements are the use of the closed depression, and the use of the interlocking grass pavers, both of which can be considered recharge and filtering practices.

Closed Depression:
2 acres of pavement drain to a stable closed depression for water quality treatment.

Area = 2.0 acres
WQv = 0.5in/12 x 2.0 ac = 0.083 ac-ft.

Interlocking Concrete Paver Blocks:
1 acre of pavement is made up of interlocking concrete paver blocks with grass openings.

Area = 1.0 acre
WQv = 0.5in/12 x 1.0 ac = 0.042 ac-ft

Adjusted WQv = 0.17 – 0.083 – 0.042 = 0.05 ac-ft

Therefore, the water quality volume that must still be accounted for in another water quality BMP is 0.05 acre-feet.

Prior to initiating design, determine the recharge volume and capture volume required as one design solution may be used for all the required volumes.

Capture Volume:

Because the pond discharges to a mild draw (no defined channel banks) located on downstream properties a capture volume is required.

Almost 100% of the site is tributary to the pond. No credits are available for the capture volume as it is a mechanism that protects downstream property owners and stream morphology in the event that all else fails. This even includes the areas draining to the closed depression, as it assumes that eventually the closed depression may fail and become permanently ponded.

60% of the site is impervious, or 10 acres.

The capture volume must equal:

$$Cv = (10 \text{ acres})\ (0.25/12) = 0.21 \text{ ac-ft.}$$

Recharge:

60% proposed site impervious = 10 acres

(Note: Since the pond is unlined, it is considered part of the pervious area.)

The recharge volume is:

$$Rv = (0.5/12)(10 \text{ acres}) = 0.42 \text{ ac-ft.}$$

In this site example, the recharge volume is greater than the water quality volume because of the use of the aluminum roofing material. However, because the site uses BMPs, several recharge credits are applicable.

Credit 1:
3 acres of pavement and 0.25 acres of sidewalk are diverted across grass buffer areas.

Credit 1 = 3.25 acres

Credit 2:
2 acres of pavement drain to a stable closed depression. Unlike water quality, the use of a closed depression is a direct credit for recharge.

Credit 2 = 2.0 acres

Credit 3:
1 acre of pavement is made up of interlocking concrete paver blocks with grass openings. Unlike water quality, a permeable surface treatment is a direct credit for recharge.

Credit 3 = 1.0 acre

Total Recharge Credits:

Total Credit = 3.25 + 2.0 + 1.0 = 6.25 acres

Therefore, the adjusted Rv = 0.5in x (10 – 6.25) acres = 1.875 acre-in = 0.16 ac-ft storage.

Therefore, the recharge volume that must be accounted for in additional recharge BMPs is 0.16 acre-feet.

Additional SWM Design Elements Required:

Since the BMPs designed into the site did not reduce the water quality, recharge, or capture volumes to zero (0), additional mitigation will be required. The remaining volumes to be treated are:

Water quality volume = 0.05 acre-feet
Capture volume = 0.21 acre-feet
Recharge volume = 0.16 acre-feet

Because a capture volume will be required in the detention basin, it would seem appropriate to use the capture volume for water quality and recharge as well (especially since the capture volume is greater than either of the other two). If the capture volume is designed to infiltrate in 72 hours naturally, it would count fully for both the remaining water quality and recharge volumes.

Alternatively, if the designer determined that the pond capture volume cannot be drained in 72 hours, a subsurface underdrain system would be required. In this case only 50% of the capture volume can apply for the recharge and water quality volumes. An easy solution would be to increase the capture volume to 0.32 acre-feet (2 x 0.16, which is the higher of the recharge and water quality volumes).

If the designer did not or could not increase the capture volume, an additional BMP would be required for recharge and water quality.

Summary:

Typical detention facility (stormwater management) and erosion and sediment pollution control volumes required for a 16.67 acre site can be considered approximately:

For stormwater management volume = 2.5 acre-feet
For erosion and sediment pollution control* volume = 3.0 acre-feet*

*Assumed using as a sedimentation basin

Because several BMPs were used on the site, the required treatment volumes for capture, water quality, and recharge were small compared to the typically required detention and erosion and sediment pollution control volumes. Addition of sufficient volume to cover the capture, water quality, and recharge requirements will only add approximately 10% to the size of a standard detention facility.

In addition, since the closed depression was maintained, it can be used to reduce the post development runoff rates and/or volumes to the pond.

EXAMPLE 2 – Large Commercial Development without BMPs

Site Description:

- A 16.67 acre site has a proposed development with 60% impervious area.
- The total proposed site impervious area is 10 acres in size which consists of:
  - The building (1.5 acres total area) roofing material is asphalt shingles.
  - All the sidewalks (0.25 acres total area) have cross slopes that allow them to drain to grass buffer strips at least equal in width to the sidewalks.
  - A small closed depression existed on the site that 2 acres of the site drained to. The closed depression was removed by paving over it.
  - The remaining impervious (building and parking) areas (9.75 acres total area) drain directly to the stormwater management pond via storm drain pipes.
- There is a single lined stormwater management pond that collects almost 100% of the proposed impervious area The pond has a design water surface elevation (WSE) foot print of 0.5 acres.
- The site is located within a sensitive area.
- The pond discharges at the property line to a mild draw located on downstream properties.
- The natural drainageway was replaced with the stormwater management pond and storm drain pipes.

Water Quality:

The total site impervious area = 10 acres or 60%. The site impervious area (SIA) for water quality = 60%.

Therefore, the $WQ_{depth} = 0.25+(0.012)\ 2.9^{(0.044*60)} = 0.42$ inches

However, since the site is in a sensitive area,

The WQ_{depth} = 0.5 inches minimum, therefore using 0.5 inches:

The WQv = is 0.5 inch x 10 acres = 5.0 acre/inch = 0.42 acre/feet storage without any credits

Had the initial computed WQ_{depth} been greater than 0.5 inches, the higher value would have been used.

Because the site uses almost no BMPs, only one water quality credit is applicable for the sidewalks.

Credit 1 (Vegetated Filter Strip/Recharge):
0.25 acres of sidewalk are diverted across grass buffer areas. The maximum allowable credit for the use of filter or recharge strips (buffer areas included) is 50% of the WQ SIA which in this case is 0.5 x 10 acres = 5.0 acres.

Credit = 0.25 acres. Therefore, the adjusted WQv = 0.5in x (10 – 0.25) acres = 4.88 acre-in = 0.41 ac-ft storage.

Therefore, the water quality volume that must be accounted for in another water quality BMP is 0.41 acre-feet.

Prior to initiating design, determine the recharge volume and capture volume required as one design solution may be used for all of the required volumes.

Capture Volume:

Since the pond discharges to a mild draw located on downstream properties a capture volume is required.

Almost 100% of the site is tributary to the pond.

60% of the site is impervious, or 10 acres.

The capture volume must equal:

$$Cv = (10 \text{ acres})\ (0.25/12) = 0.21 \text{ ac-ft.}$$

Recharge:

60% proposed site impervious = 10 acres

(Note: Because the pond is lined, it is considered part of the impervious area.)

The recharge volume is:

$$Rv = (0.5/12)(10 + 0.5 \text{ acres}) = 0.44 \text{ ac-ft.}$$

Only one recharge credit is applicable for the sidewalks.

Credit 1:
0.25 acres of sidewalk are diverted across grass buffer areas.

$$\text{Credit} = 0.25 \text{ acres}$$

The adjusted Rv = 0.5in x (10 +0.5 – 0.25) acres = 5.13 acre-in = 0.43 ac-ft storage.

Therefore, the recharge volume that must be accounted for in additional recharge BMPs is 0.43 acre-feet.

Additional SWM Design Elements Required:

Since the BMPs designed into the site did not reduce the water quality, recharge, and capture volumes to zero (0), additional mitigation will be required. The remaining volumes to be treated are:

Water quality volume = 0.41 acre-feet
Capture volume = 0.21 acre-feet
Recharge volume = 0.43 acre-feet

Because a capture volume will be required in the detention basin, it would seem appropriate to use the capture volume for water quality and recharge as well. However, because the SWM pond is designed in a drainage-way, it most likely will not be able to meet the 72 hours infiltration requirement naturally. If this is the case, a subsurface underdrain system could be used. In this case only 50% of the capture volume would apply for the recharge and water quality volumes. The solution would require a capture volume of 0.86 acre-feet (2 x 0.43, which is the higher of the recharge and water quality volumes). The magnitude of this treatment volume may dictate that a secondary facility (such as an infiltration trench or bed) be used to provide the necessary volume.

Summary:

Since almost no low impact BMPs were used as a part of the proposed site development, the required treatment volumes are significantly larger than those required in the previous example. In this case, additional structural BMPs will be required to address water quality and recharge requirements.

In addition, the removal of the closed depression will have a major effect on peak runoff rate computations. This is because the 2 acres initially tributary to the closed depression may not be added into the pre-development total area and runoff computations, but must be included in the post-development runoff analysis. This will result in larger post-development flows and a larger stormwater detention volume.

EXAMPLE 3 – Large Commercial Development Draining to Sinkhole

Site Description:

- A 16.67 acre site has a proposed development with 60 % impervious area.
- A sinkhole exists on the site that takes all surface runoff from the site.
- There is no defined downstream conveyance system and a residential development was previously constructed in the downstream low ground.
- The total proposed site impervious area is 10 acres in size which consists of:
    - The building (1.5 acres total area) roofing material is asphalt shingles.
    - All the sidewalks (0.25 acres total area) have cross slopes that allow them to drain to grass buffer strips at least equal in width to the sidewalks.

Peak Runoff Rate Control:

Since the total site area drains to a sinkhole, the need for a stormwater management pond may be questioned. However, sinkholes, like any closed conduit, have a limiting capacity and therefore accept runoff at a given maximum rate. Therefore, a stormwater management pond should almost always be constructed upslope to control the rate of discharge to the sinkhole.

More importantly, any change in the hydrologic response to a sinkhole may cause adverse conditions, i.e., further collapse, or the rejection of all surface runoff. Both of these conditions have occurred frequently in the Spring Creek Watershed. A qualified hydrogeologist must assess the stability and risk of using any sinkhole for stormwater discharges. Recall that development will always change (increase) the frequency and volume of surface runoff.

In some cases, such as this, where there is no downstream conveyance system, if the sinkhole were deemed unstable, or at a high risk to cause contamination to groundwater, the Municipality would need to make the decision if the property should be developable considering the health and safety of the community.

In addition, a plan should be in place to address the possibility that the sinkhole may start rejecting surface runoff inputs at some point in the future. This plan must consider a mechanism for conveying runoff downstream. For this reason, development should be restricted in drainageways even when upstream sinkholes may be currently capturing all upslope runoff.

Sealing or bypassing the sinkhole should also be discouraged because this almost always induces additional downstream surface flooding.

Water Quality:

The total site impervious area = 10 acres or 60%. The site impervious area (SIA) for water quality = 60%.

Therefore, the $WQ_{depth} = 0.25+(0.012)2.9^{(0.044*60)} = 0.42$ inches

However, since the site discharges to a sinkhole, the Municipality has the right to request a higher standard of water quality. However, we will assume that this is not the case here.

The WQv = is 0.42 inch x 10 acres = 4.2 acre/inch = 0.35 acre/feet storage without any credits.

Had the initial computed WQ_{depth} been greater than 0.5 inches, the higher value would have been used.

Because the site uses almost no BMPs, only one water quality credit is applicable; for the sidewalks.

Credit 1 (Vegetated Filter Strip/Recharge):
0.25 acres of sidewalk are diverted across grass buffer areas. The maximum allowable credit for the use of filter or recharge strips (buffer areas included) is 50% of the WQ SIA which in this case is 0.5 x 10 acres = 5.0 acres.

Credit = 0.25 acres Therefore, the adjusted WQv = 0.42in x (10 – 0.25) acres = 4.10 acre-in = 0.34 ac-ft storage.

Therefore, the water quality volume that must be accounted for in another water quality BMPs is 0.34 acre-feet.

Prior to initiating design, determine the recharge volume and capture volume required as one design solution may be used for all of the required volumes.

Capture Volume:

Because the pond discharges to a sinkhole, a capture volume is not specifically required.

Recharge:

Because the site runoff is discharged directly into a sinkhole, recharge occurs at 100%. However, any BMPs that can be used, should be used to reduce the amount of additional surface runoff entering the sinkhole.

Summary:

Peak runoff rate control is still required (peak rate comparison). However, water quality protection is the most important concern for this site and while the capture volumes and recharge volumes are not specifically required, BMPs should be employed that will both protect the water quality and induce recharge. The required volumes with credits were:

Water quality volume = 0.35 acre-feet
Capture volume = 0.0 acre-feet
Recharge volume = 0.0 acre-feet

EXAMPLE 4 – Small Water Quality Sensitive (WQS) Development

Site Description:

- A 2.0 acre site has a proposed development with 70% impervious area.
- The proposed land use is a gas station.
- The site is located within a sensitive area.
- The site would discharge to an existing storm drain system that has no past history of flooding.

Water Quality:

The total site impervious area = 1.4 acres or 70%.

Therefore, the $WQ_{depth} = 0.25+(0.012)\ 2.9^{(0.044*70)} = 0.57$ inches

The WQv = is 0.57 inch x 1.4 acres = 0.80 acre/inch = 0.07 acre/feet storage.

Because the site is a water quality sensitive (WQS) development in a sensitive area, no water quality credits are permitted.

Capture Volume:

The stormwater management detention facility, whether a surface pond or subsurface vault, would not need a capture volume. In addition, if the site design engineer could show that the additional 100-year runoff caused no undue harm, the detention facility may be allowed to leave the 100-year runoff rate uncontrolled. The final decision would be the decision of the Municipal Engineer.

Recharge:

70% proposed site impervious = 1.4 acres

The required recharge volume is:

$$Rv = (0.5/12)(1.4 \text{ acres}) = 0.06 \text{ ac-ft.}$$

Additional Notes:

The water quality features need to be designed so runoff goes through a pre-treatment (non-infiltrating) BMP prior to being recharged. In addition, the site designer should ensure that no undue harm will occur to adjacent properties as a result of recharging runoff on a small portion of a small site.

EXAMPLE 5 – Commercial Development with Possible Recharge Exemption

Site Description:

- A 5.0 acre site has a proposed development with 70% impervious area.
- The proposed land use is general commercial.
- The site is not located within a sensitive area.
- The site would discharge to an existing well defined storm drain system that has a past history of flooding.
- The site is located on land with an average slope of 10%.
- The downslope properties are already developed with buildings that have basements and are within 20 feet of the property line.
- Because of the slope, the entire site will need to be graded for the development.

Water Quality:

The total site impervious area = 3.5 acres or 70%.

Therefore, the $WQ_{depth} = 0.25+(0.012)\ 2.9^{(0.044*70)} = 0.57$ inches.

The WQv = is 0.57 inch x 3.5 acres = 2.0 acre/inch = 0.17 acre/feet storage.

Capture Volume:

Because the pond discharges to a well defined storm drain system, a capture volume will not be required even though there has been a past history of flooding. This is because the capture volume will have little impact on an overtaxed storm drain system, unless the system floods during most rainfall events. If this is the case, it would serve in the community's best interest for the Municipality to require over-detention, which could be done using a concept similar to the capture volume.

Recharge:

70% proposed site impervious = 3.5 acres

The required recharge volume is:

$$Rv = (0.5/12)(3.5\ \text{acres}) = 0.15\ \text{ac-ft.}$$

However, there is a probability that if the surface runoff generated from 3.5 acres of impervious area are infiltrated into a small area, the infiltrated water could move laterally towards the downstream building causing flooding of the basements. In this case, it would be appropriate for the site developer to have a qualified hydrogeologist determine if recharge should be exempted.

Additional Notes:

The final decision for any stormwater exemption lies with the Municipal Engineer. In some cases the Municipality may argue that the site should not be developed to as high a density regardless of the zoning.

EXAMPLE 6 – Re-development Example in a Highly Urban Area

Site Description:

- A 4.0 acre site has 3 acres of existing impervious area.
- A developer wishes to redevelop the site with only 2.5 acres of impervious area.
- The site in its existing condition does not have any stormwater management facility.
- The site discharges to an existing storm drain system that has adequate capacity in the vicinity of the development, but past history of flooding has occurred 0.5 miles downstream.

In accordance with exemption D (Chapter 3 of the Plan and Section 110 of the Model Ordinance) the Municipal Engineer has the right to waive the requirement for submission of a formal stormwater management plan since the impervious area on the site is being reduced.

Water Quality:

The total proposed site impervious area = 2.5 acres or 62.5%.

Since the developer is planning to remove 0.5 acres (or 17% of the total impervious area) the Municipality has the right to waive additional water quality criteria unless the site is a Water Quality Sensitive (WQS) development or is located in a recognized sensitive area.

Capture Volume:

Because the existing site and the area directly downstream do not have any stormwater flooding problems, no stormwater management facility (other than storm drains) should be required. Making a small site such as this provide detention to prevent flooding further downstream is unrealistic and may be counterproductive to redevelopment.

Recharge:

Even though the site has 62.5% impervious area, the Municipality has the right to waive the recharge criteria. This is because the redevelopment will already improve recharge by removing 0.5 acres of impervious area.

Additional Notes:

While the Municipality may choose to exempt peak runoff rate control, water quality, recharge, and/or capture depth criteria, the site designer still has the obligation to design the conveyance system to protect the health and safety of the public at large.

If the site were a WQS development or located in a sensitive area, the site could not be exempted from the requirement to prepare a stormwater plan, or evaluate the water quality volume. However, if the site was in a sensitive area, but was not a WQS development, the Municipal Engineer could allow creative solutions, which would provide the necessary water quality protection.

EXAMPLE 7 – Small Site with Downstream Flooding Problems

Site Description:

- A 0.75 acre site has 0.65 acres of existing impervious area.
- A developer wishes to redevelop the site with only 0.5 acres of impervious area.
- The site in its existing condition does not have a stormwater management facility.
- The site is not a water quality sensitive (WQS) development or located in a sensitive area.
- The site discharges to an existing storm drain system that has a past history of flooding at the site and the area directly downstream.

Because of the past history of flooding, the site may not be exempted from the requirement to prepare a stormwater management plan per Exemption C.

Water Quality:

Since the developer is planning to remove 0.15 acres (or 23% of the total impervious area) the Municipality has the right to waive additional water quality criteria.

Capture Volume:

Because the existing site does not have a stormwater facility, no capture depth is required.

Recharge:

Even though the site has 66.7% impervious area, the Municipality has the right to waive the recharge criteria. This is because the redevelopment will already improve recharge by removing 0.15 acres of impervious area.

Additional Notes:

The site developer would be required to submit a stormwater management plan for this redevelopment. However, it would consist simply of verification of the impervious area reduction and storm drain capacity.

Even though the site has flooding problems, historically these types of redevelopments have not been required to conduct peak runoff rate control. The flooding and land use rights are grandfathered. This same logic should apply to water quality, recharge, and the capture volume requirements under the stated conditions. However, in the interest of community relations, the developer may have an ethical obligation to correct or minimize the problems as much as possible. However, if for example the site were located in a large highly developed area and further control would realistically have no impact on downstream flooding, the Municipality has the right to waive the stormwater management plan requirements.

If the site were a WQS development or located in a sensitive area, the Municipality could require that water quality BMPs be employed. However, if the site was only in a sensitive area, but was not a WQS development, the Municipal Engineer could allow creative solutions, which would provide the necessary water quality protection.

EXAMPLE 8 – Progressive Site Design Promoting Infiltration

Site Description:

- A 5.0 acre site has a proposed development with 70% impervious area.
- The proposed land use is general commercial.
- The site is not located within a sensitive area.
- The site engineer claims that they can design the site with zero increase in peak runoff rates or volumes by using an infiltration bed under the parking lot for all impervious areas.

Water Quality:

The total site impervious area = 3.5 acres or 70%.

Therefore, the $WQ_{depth} = 0.25+(0.012)\ 2.9^{(0.044*70)} = 0.57$ inches.

The WQv = is 0.57 inch x 3.5 acres = 2.0 acre/inch = 0.17 acre/feet storage.

The site engineer would need to show that this infiltration bed is effective for water quality (refer to additional notes section).

Capture Volume:

Because the infiltration bed assumptions claim that no discharge would occur for 0.25 inches of precipitation, no capture volume would be required.

Recharge:

70% proposed site impervious = 3.5 acres

The required recharge volume is:

$$Rv = (0.5/12)(3.5\ \text{acres}) = 0.15\ \text{ac-ft.}$$

The site engineer claims that the recharge far exceeds this volume. Supporting computations would be required.

Additional Notes:

With these types of systems, water quality and the potential for sinkhole formation are of greatest concern. If the site is designed such that the infiltration bed bottom is located on or near bedrock, the Municipality has the full right to reject the design from a water quality perspective and may require pre-treatment BMPs to be employed. This is because gravel beds have very little water quality benefit. The upper soil horizons O and A that form the topsoil and the B horizon (where active soil forming processes are occurring) are the only parts of the soil that can be counted on for long term self sustaining/renovating water quality benefit.

In addition, the municipality may require a higher level of quality control under these conditions. Therefore, the site designer is encouraged to maintain as much natural insitu soil as possible over the underlying bedrock.

EXAMPLE 9 – Pond Capture Volume Dewatering Calculation Methods

The pond capture volume is a volume of runoff that will be retained in a pond below the elevation of any free surface principal spillway orifice. No principal spillway orifice (except those connected to subsurface drains, or maintenance plugs), regardless of how small, shall be below the pond elevation equivalent to this volume.

The capture volume will be allowed to infiltrate, evaporate, or dewater from a subsurface drain system connected directly to the facility's principal spillway. Supporting computations that show that 90% of the capture volume can dewater in a maximum of 72 hours must be provided. Designs that rely on the natural infiltration of insitu soils must provide computational support for the assumed infiltration rates.

To simplify computational requirements for design event analysis, designers do not need to calculate discharges from subsurface drains related to the capture volumes if the filter media is sand or material smaller than AASHTO 57 stone.

The Capture Volume in a pond MUST be assumed to be full at the start of design event routing in the Hydrologic Model or with TR-55. The Capture Volume therefore may NOT be used to reduce pond peak discharges.

If the capture volume is allowed to infiltrate through a natural soil profile or exfiltrate through the use of a subsurface unit such as an infiltration bed or trench, then a measure of recharge will occur in the control area. However, if the designer uses a subsurface under drain to dewater the capture volume from a pond, recharge does not occur to as high a level (and may not occur at all) in the pond. However, the concept of a capture volume using a subsurface under drain is based on the assumption that the discharge rate will be so low that the discharged runoff can be re-infiltrated further downstream (a very real natural effect with karstic soils).

Example Calculations:

The following examples define the computational procedures that must be used for documenting the capture volume.

If a 10 acre site has 2 acres of impervious area (regardless if connected or unconnected) and has a stormwater management pond, the required capture volume will be calculated as:

Required minimum capture depth per acre imperviousness = 0.25 in/ac
Required volume = (2 x 43560) x (0.25/12) = 1,815 cubic feet = 0.0417 ac-ft

The pond stage storage distribution needs to be defined and integrated into the design. Assume the following stage/storage data:

Elevation	Storage (ac-ft)
1000.0	0.000
1001.0	0.025
1001.5	0.060
1002.0	0.140

The volume of 0.0417 ac-ft can be interpolated from the data to produce a capture depth equivalent to an elevation of 1001.24 (or a capture depth of 1.24 feet). Since this is less than 18 inches, the capture depth may be conservatively set at elevation 1001.5 (18 inches capture depth). This would result in a capture depth greater than 0.25 in/ac, which is acceptable.

Using Natural Infiltration

Ponds that attempt to use the soils infiltration rate for dewatering the capture volume must protect the soils from construction disturbance and/or compaction (preferred). If this cannot be accommodated the soils must be renovated. The final infiltration rate determined from insitu measurements must be used for the computations.

The design professional is permitted to assume a uniform rate over the falling head of the capture volume. The dewatering time is to be computed using continuity relationships. This can be implemented using the Modified Puls Routing Method, whereby the pond stage/storage is defined based on the capture volume geometry (as indicated above), and the stage/discharge rate is defined from the final infiltration rate.

The stage discharge will be computed as follows:

If the average measured infiltration capacity is 50mm/hr = 1.97 in/hr, convert to a rate of feet/sec.

1.97 in/hr = 0.0000455 ft/sec, multiply this rate times the capture volume surface area at each stage. The result is in units of volume/time.

The resulting dewatering time must be less than 72 hours.

Using Engineered Infiltration (Exfiltration)

Systems that exfiltrate runoff into the subsoil will be essentially computed the same as for infiltration except that the percolation rate or saturated hydraulic conductivity shall be used to define the exfiltration rate instead of the surface infiltration capacity.

A field measured percolation rate for the soil at or near the bottom of the exfiltration bed or trench must be used for this calculation. In addition, only the bottom surface area of these systems may be used as the area across which exfiltration occurs (the sides of trenches may not be used).

The following example shows why the use of published infiltration rates, saturated hydraulic conductivity, percolation rates, or soil permeability coefficients may NOT be used for design of these systems.

If it is assumed that a gravel bed is filled with water, and the time that it takes to dewater using information from the Soil Survey needs to be calculated, the following differences in times can easily result:

Assume a gravel bed bottom area = 360 square feet
Assume the volume of ponded water in the gravel is 2000 cubic feet
Assume that the flow rate through the gravel is non-limiting.
Assume the designer wishes to use a published value for the permeability coefficient, K from a Soil Survey. Assume the soil is a Hagerstown Silt Loam. At the intended design depth of the gravel bed, K varies from 0.6 in/hr to 2.0 in/hr.

If we assume that the time to dewater can be calculated as:

$$Td = \frac{\text{volume of ponded water}}{\text{(gravel bed bottom area)(Permeability Coefficient)}}$$

For a K = 0.6 in/hr, the time to dewater = 111 hours
For a K = 2.0 in/hr, the time to dewater = 33 hours

One computation is less than 72 hours, while one is greater than 72 hours. Therefore in order that a design professional does not need to choose a design value, insitu measurements must be made.

Using a Subsurface Drain

If subsurface drains are used to underdrain the capture volume, a dewatering time does not need to be calculated. However, the USDA NRCS Engineering Field Manual design procedures for subsurface drains (or tile fields) should be used as a guideline in designing the underdrain system.

EXAMPLE 10 - Development Example Using Historic Runoff Data

Examples abound in literature documenting the implementation of BMPs into low impact developments for residential developments. However, almost nowhere can one find examples documenting how to develop large commercial sites (with large percentages of impervious area coverage) using these BMPs. This example documents approaches and calculation methodologies for a large development using both traditional methods and innovative best management techniques. The results and consequences of each are discussed.

This example uses a research watershed with measured pre-development precipitation and runoff data as a development site. This provides the reader with an understanding of actual pre development to post-development impacts in both cases. Portions of the following example are from Fennessey et al. [2001, *Changes in Runoff Due to Stormwater Management Pond Regulations*, Journal of Hydrologic Engineering. 6(4):p317-327].

Watershed History and Description:

The watershed used for this following example is an actual research watershed used by the USDA, ARS and NRCS located on the Southern Piedmont Conservation Experiment Station near Watkinsville, Georgia. The watershed is 19.2 acres in size, consists of 100 percent of hydrologic soil group (HSG) B soils, has a slope range of 3 percent to 10 percent, and has an average slope of 7 percent. A map of the actual watershed can be seen in Figure C-1.

Although the site is located in the southeastern United States, it was chosen for this example for several reasons. First, the watershed's size and shape make it an ideal candidate for a small commercial development. Second, the hydrologic model used (SCS's TR-55) reasonably predicted the pre-development peak runoff rates, thereby eliminating the potential of nuisance flooding due to an over prediction of the runoff by a model. Third, the watershed soils are comprised of a single NRCS hydrologic soil group. Fourth, the watershed cover was good continuous pasture for 33 years (from 1947 to 1979). Fifth, the watershed data are well documented, with both runoff and precipitation data in continuous form. And finally, the period of record contained no snowfall or snowmelt events.

Hypothetical Site Development:

The site was hypothetically developed using designs with 50, 60, and 70% impervious coverage (impervious area includes the building square footage, parking areas and drives). These impervious areas' percentages were selected because of their common use in zoning ordinances for commercial developments. The pond was considered a part of the 30% pervious area. The pond was not considered to have an impervious lining and was allowed to provide an abstraction for surface runoff from the impervious areas. The site was graded such that all surface runoff was directed to the pond via overland flow over pavement or through storm drains. A traditional 70% impervious area site design is shown in Figure C-2. Additional site design criteria are as follows:

- Site was graded with an average slope of 3.5% in the parking area.
- Site earthwork (cut and fill) was balanced; this balancing was used to set site elevations.
- Two hypothetical off-site access roads were assumed to exist along the southern boundaries.
- Any up-slope, off-site runoff was assumed to be diverted around the site.
- Only one stormwater management pond was used.
- Pond abutments had a combined slope of 5:1 and were 10 feet wide at the crest.
- Emergency spillway elevation was set at the peak 100-year water surface elevation.
- Pond bottom slope was maintained at 2%.
- Pond stage/storage relationship was the same for each site.

Site Development using the Traditional Approach:

The watershed was modeled as prescribed in this Stormwater Management Plan (recognizing the soils are not underlain by carbonate geology and Georgia precipitation depths were used). The comparison of the historical and pre-development partial series runoff rates (using the model ordinance herein) are:

Return Period	Historical Peak Runoff Rate (cfs)	Computed Pre-Development Peak Runoff Rate (cfs)	TP-40 (24-hr) Precipitation (in)
1-year	5.0	4.7	3.2
2-year	9.9	9.2	3.8
5-year	17.4	18.0	4.75
10-year	25.5	25.6	5.5
25-year	41.4	37.0	6.5
50-year	55.8	49.1	7.5

It can be observed that the model estimates vary slightly from the observed "historical" runoff values. However, it must be noted that these estimates are extremely good (normal hydrologic estimates within a ±30% range are typically considered good). Using the 70% traditional development example shown in Figure C-2 (no BMPs, recharge, capture, or water quality volumes), the following changes in peak runoff are predicted:

Return Period	Post-Development Uncontrolled Peak Runoff Rate (cfs)
1-year	58.0
2-year	74.4
5-year	101.0
10-year	122.4
25-year	151.4
50-year	180.7

The above runoff rates would be those that were directed to the stormwater management pond. If no stormwater management were provided, these would be realistic discharges from the site

following development. However, the pond shown in Figure C-2 could easily be designed to control the differences in the peak runoff rates as is traditionally done using stormwater management ponds. However, there would still be radical changes in the hydrology of the site.

The historical average annual precipitation depth (from 1947 to 1979) was 49.8 inches. The historical average annual runoff volume (in inches from 1947 to 1979) was 2.83 inches. Continuous modeling simulations for the site have shown that had the site been developed as shown in Figure C-2 the average annual runoff volume (in inches from 1947 to 1979) would have been 21.64 inches or an increase of 764%. In addition, there would have been a radical increase in the number of small runoff events (less than a 1-year return period) from the pond because of the large addition of impervious area.

In an attempt to create a low impact site design a developer could propose to simply reduce the site's impervious area. Figure C-3 shows a traditional development that has been designed for the site using a 50% impervious area. Although it might be argued that this is a low impact design, the hydrologic changes that would occur are very similar to the 70% example. True low impact designs must include water quality, capture, and recharge volumes. However, if these considerations are implemented (as required by this PLAN) significant improvement in hydrologic response can be achieved, and the stormwater management basin or pond would not have to be increased in size significantly.

Site Development implementing the PLAN Standards:

In addition to traditional peak control standards used in the developments illustrated in Figures C-2 and C-3, the technical standards outlined in Chapter 3 and Appendix A (Model Ordinance) of this PLAN outline additional peak control requirements as well as standards for water quality, capture, and recharge. A design is developed in this portion of the example, which complies with PLAN standards. Because there is no distinct channel at the outlet, a capture volume will need to be used in the pond design. In addition, water quality and recharge considerations will need to be met. These additional criteria do not limit the site design, or the impervious coverage the developer wishes to use. The site could be increased from 50% impervious area (as previously shown in Figure C-3) to 60% or greater impervious area. A 60% impervious site design can be seen in Figure C-4. A recharge basin has been designed for this site that can also be counted as a closed depression storage area for modeling purposes. Calculation methodologies are presented below for this design.

The stage/storage relationship for the pond shown in all of the figures is:

Elevation (ft)	Area (acres)	Storage (ac-ft)
708	0.00	0.00
710	0.21	0.21
712	0.78	1.20
713	1.04	2.10
714	1.09	3.16
715	1.14	4.28

Routing the 25-year return period event for the traditional site design (70%) in Figure C-2 would result in a maximum water surface elevation of approximately 714.2 feet, or a storage volume of

approximately 3.2 acre-feet (keep in mind that the design precipitation depths in Watkinsville, Georgia are nearly double those of Centre County, Pennsylvania).

For a site with 60% impervious area (11.52 acres) with an asphalt-based roof:

Recharge Criteria:

60% proposed site impervious
For a 19.2 acre site, the recharge volume is:

$$Rv = (0.5/12)(19.2x0.60 \text{ acres}) = 0.48 \text{ acre-feet}$$

This volume of dead storage could be designed in the closed depression shown in Figure C-4. It is assumed that roof leaders could be diverted to this closed depression. The closed depression should be less than one (1) foot deep at its deepest point and does not require an outlet structure or emergency spillway. Runoff events greater than the storage capacity of the closed depression would be directed towards the pond via overland flow. The volume of storage for the closed depression could also be used to adjust the water quality and capture volumes, and the design event runoff.

Allowing runoff to flow across the lawn areas directly adjacent to the pond can not be used as a recharge credit since the area is in the immediate proximity of the pond.

Water Quality Criteria:

$$\text{The } WQ_{depth} = 0.25+(0.012)2.9^{(0.044*60)} = 0.45 \text{ inches}$$

Since the pond is not in a karst area the minimum WQ_{depth} = 0.5 inches

$$\text{The } WQ_v = (0.5 \text{ inch})/12 \text{ x } 11.52 \text{ acres} = 0.48 \text{ acre-feet of storage}$$

Since 0.48 acre-feet of runoff is diverted into the closed depression area for recharge, this area can be credited completely against the water quality volume. The pond capture volume could have also been used as a water quality credit if required.

Capture Volume:

Since the pond outlets in a mild draw, a capture volume is required. Since 100% of the site is tributary to the pond, the impervious area = 11.52. However, if we assume that 1.92 acres of the building roof is directed to the closed depression, we could remove the roof area from the capture volume. Therefore, 11.52 – 1.92 = 9.6 acres. The pond capture volume must be:

$$Cv = 9.6 \text{ acres x } (0.25\text{inch}/12) = 0.2 \text{ acre-feet of storage}$$

This volume is required in the pond regardless of any other site BMPs. However, it could have been used as a credit against other criteria.

Figure C-5 presents a site that includes several additional stormwater BMPs including two closed depression storage areas, a water quality BMP, a pond with a capture volume, and undisturbed natural areas.

Finally, and most importantly, the peak runoff rates to the pond may be adjusted because of the closed depression storage volume. In order to do this several procedures could be used. Approaches for adjusting peak runoff rates for closed depression storage are considered in Example 11.

EXAMPLE 11 – Calculation Methods for Closed Depressions and Engineered Infiltration

The following example illustrates several different methods that can be used to evaluate closed depressions and assess engineered infiltration BMPs. This is not intended as a complete list; other approaches may be appropriate depending on site specific conditions. However, engineers and designers are cautioned to fully understand the assumptions, limitations, math, and logic of any method they are applying.

Method 1 - Adjusted CN Volume

Peak Rates can be adjusted for natural or engineered closed depressions. The following example shows how the site in Example 10 (Figure C-4) could be adjusted.

For this example we will assume that the engineered closed depression had a storage volume of 0.24 acre-feet. This assumes that other BMPs were also used.

The post-development hydrographs are traditionally computed as a function of area, curve number, time of concentration, precipitation depth based on some frequency, precipitation distribution, and unit hydrograph. Closed depression areas such as those shown in Figure C-4 can be used to adjust the post-development CN volume as illustrated in the following example.

Traditional CN computation:

Land Use	CN	Area (acres)
Grass	61	7.68
Impervious	98	11.52

Weighted CN = 83

The weighted CN can be adjusted by volume. Since the CN = 83 is for an area of 19.2 acres, it represents a runoff volume that can be calculated as:

$CN = 1000/(S+10)$

If CN = 83, S can be solved as S = 2.05

S can then be used to compute the runoff volume solving the equation:

Runoff Volume, $Q = (P-0.2S)^2/(P+0.8S)$

where: P = the design event precipitation depth in inches
Q = the runoff volume in inches

For our 10-year event:

$Q = (3.6 – 0.2 \times 2.05)^2/(3.6 + 0.8 \times 2.05) = 1.94$ inches

Q volume = 1.94 /12 x 19.2acres = 3.11 acre-feet

The closed depression volume could now be subtracted from this volume.

Adjusted Q volume = 3.11 acre-feet – 0.24 acre-feet = 2.87 acre-feet

Adjusted Q = 2.87 acre-feet x (12/19.2) = 1.79 inches

The equivalent Adjusted S is than computed as:

$$\text{Adjusted S} = 5(P + 2Q - (4Q^2 + 5PQ)^{0.5})$$

$$\text{Adjusted S} = 5(3.6 + 2 \times 1.79 - (4 \times 1.79^2 + 5 \times 3.6 \times 1.79)^{0.5}) = 2.34$$

The Adjusted S is then converted into an adjusted CN

Adjusted CN (for a 10-year return period) = 1000/(S+10) = 81

This adjustment **must** be done for each design event precipitation depth. In addition, no adjustments in the time of concentration are to be made and the adjusted CN **cannot** be used in the NRCS lag equation. The adjustment can only be done for runoff routed to a stormwater detention pond. If the designer does not fully understand this concept, it should not be used. An example of results in using this procedure is explained below.

The impact of the adjustment outlined above can be documented by example. This example uses the stage/storage data previously defined for the site, and a Tc of 14.4 minutes. The routed 10-year Centre County storm data areas follows:

CN	Tc (min)	Area (acres)	P (in)	P (dist)	Qp to pond (cfs)	Runoff (in)
83	14.4	19.20	3.6	II	41.4	1.94
81	14.4	19.20	3.6	II	37.5	1.79

Method 2 – Modeling Engineered Infiltration Beds

Computations can be conducted in one of the two methods outlined in the following:

1. **Engineered Infiltration Beds as Reservoirs -** Best management practices such as porous pavement, or infiltration trenches may be modeled as ponds. Once the pond volumes are full, the overflow is based on a traditional stage/storage/discharge relationship. In this case the systems are modeled as Ponds or reservoirs. This is the simplest method to use and is conservative. In this instance, the rate of discharge is reduced and a runoff volume equal to the captured volume is removed from the surface water runoff hydrograph. An example of how this would be modeled is shown below. As indicated, the reservoir has no effective discharge until elevation 1089.05 when it spills over. At this elevation a volume of 0.290 ac-ft would have been captured. (Note: Discharge should always increase as shown. Rates of discharge of 0.001cfs and 0.002cfs, etc., are considered zero.) The design professional now may compute the time to dewater, if required.

Elevation (ft)	Discharge (cfs)	Storage (ac-ft)
1087.0	0.000	0.000
1088.0	0.001	0.036
1089.0	0.002	0.265
1089.05	5.410	0.290
1089.1	17.330	0.317
1089.2	56.100	0.371

2. **Engineered Infiltration Beds as Diversions – These methods can only be used in more sophisticated models (TR-020 and HEC-HMS for example) that allow losses to be assessed as diversions.** Best management practices may also assume that substantial infiltration occurs during the design event. In such a case the systems can be modeled as a diversion, whereby the peaks and volumes are removed from surface runoff at a defined rate. These assumptions should not be used for extreme (50- or 100-year) runoff events in the Spring Creek Watershed since these events usually occur as a result of major snowmelts. Also, designers are cautioned not to create high subsoil exfiltration rates as a part of BMP design. High exfiltration rates result in high soil pore velocities, which could result in sinkhole development.

The following examples define what computational procedures could be used for engineered infiltration (natural or enhanced infiltration and subsoil exfiltration) that attempt to define the losses as diversions.

Infiltration Losses in Swales and Channels

1. Determine the average channel flow parameters: average top width of the flow (not channel), length of channel, mannings coefficient, average velocity.
2. Using these average values compute the average travel time in the channel.
3. Determine the final infiltration rate of the soils in the channel.
4. Compute the infiltrated depth of runoff in the average travel time.
5. Multiply the infiltrated depth by the channel average top width and channel length to determine an equivalent loss in cfs.
6. Model this loss in cfs as a steady state loss in a model as a diversion.

An example follows:

A natural channel exists where the average top width is 50 ft and the channel length is 1120 ft. The average velocity is computed to be 1.7 fps.

The time for a plug of runoff to move down the channel is : 1120 ft/1.7 fps = 658.8 sec.

Assume that the final infiltration rate was found to be 2.0 in/hr = 0.00056 in/sec = 0.0000463 ft/sec

The depth of infiltrated runoff in 658 sec would be (0.0000463 ft/sec) x (658.8 sec) = 0.0305 ft

The volume of runoff infiltrated across the channel would be:

Vol = (0.0305 ft) x (50ft) x (1120ft) = 1708 cubic feet

The average steady state loss would then be:

Qloss = 1708 cf/658.8 sec = 2.60 cfs, which is the value placed in the model as the steady state diverted flow discharge

Infiltration Losses in Constructed Closed Depressions

To use an infiltration loss in a closed depression as a diversion the following must apply:

1. The existing soil profile is left undisturbed in the proposed pond (i.e., no grade changes) or slightly disturbed soils are renovated.
2. The existing and proposed land surface cover must be a stable vegetative cover.
3. The closed depression and ponding is to be created through the use of raised weirs or abutments that make the area mimic a closed depression.
4. The final infiltration rate will be used for design purposes.

The table below illustrates the type of data that will need to be defined for this procedure. The design professional must determine the bed infiltration rate over the closed depression or pond plan area, then determine what the overflow system capacity is. In the example below, column two represents the computed infiltration rate. This rate is computed from the measured infiltration rate of the soil times the depression surface area at the particular elevation (see Example 9). Note that the resulting infiltration rate increases as the ponding surface areas increase. At elevation 1089.05 overflow occurs, but infiltration still continues.

Elevation (ft)	Infiltration Rate (cfs)	Overflow Discharge (cfs)	Storage (ac-ft)
1087.0	0.000	0.000	0.000
1088.0	2.200	0.000	0.036
1089.0	2.600	0.000	0.265
1089.05	2.800	5.000	0.290
1089.1	3.000	10.000	0.317
1089.2	4.000	20.000	0.371

This condition can then be modeled as a rating curve diversion in the model. All computed values and assumptions and documentation must be provided by the design professional using this method.

Subsoil Exfiltration (Infiltration Beds/Trenches, Porous Pavement)

Subsoil exfiltration differs from infiltration by two elements: 1) the method used to determine the rate of movement through the subsurface; and 2) the surface area of exfiltration does not change so the exfiltration rate remains constant as indicated in the following table:

Elevation (ft)	Infiltration Rate (cfs)	Overflow Discharge (cfs)	Storage (ac-ft)
1087.0	0.000	0.000	0.000
1088.0	2.200	0.000	0.036
1089.0	2.200	0.000	0.265
1089.05	2.200	5.000	0.290
1089.1	2.200	10.00	0.317
1089.2	2.200	20.00	0.371

This condition may be modeled as a rating curve diversion in a model; however, the simpler method is to divert the flow without a rating curve. All computed values and assumptions and documentation must be provided by the design professional using this method.

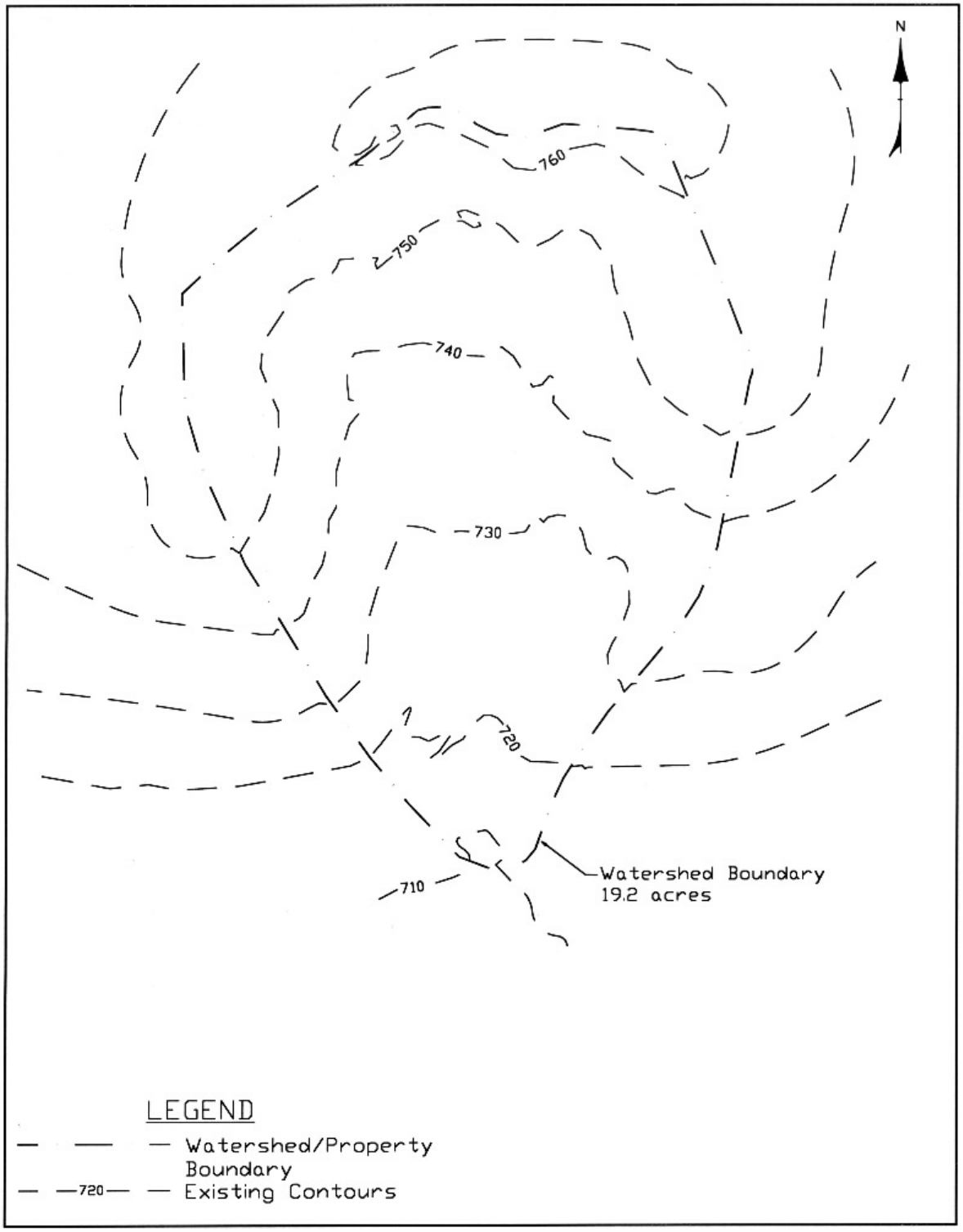

Figure C-1

Example Sketch – Existing Conditions

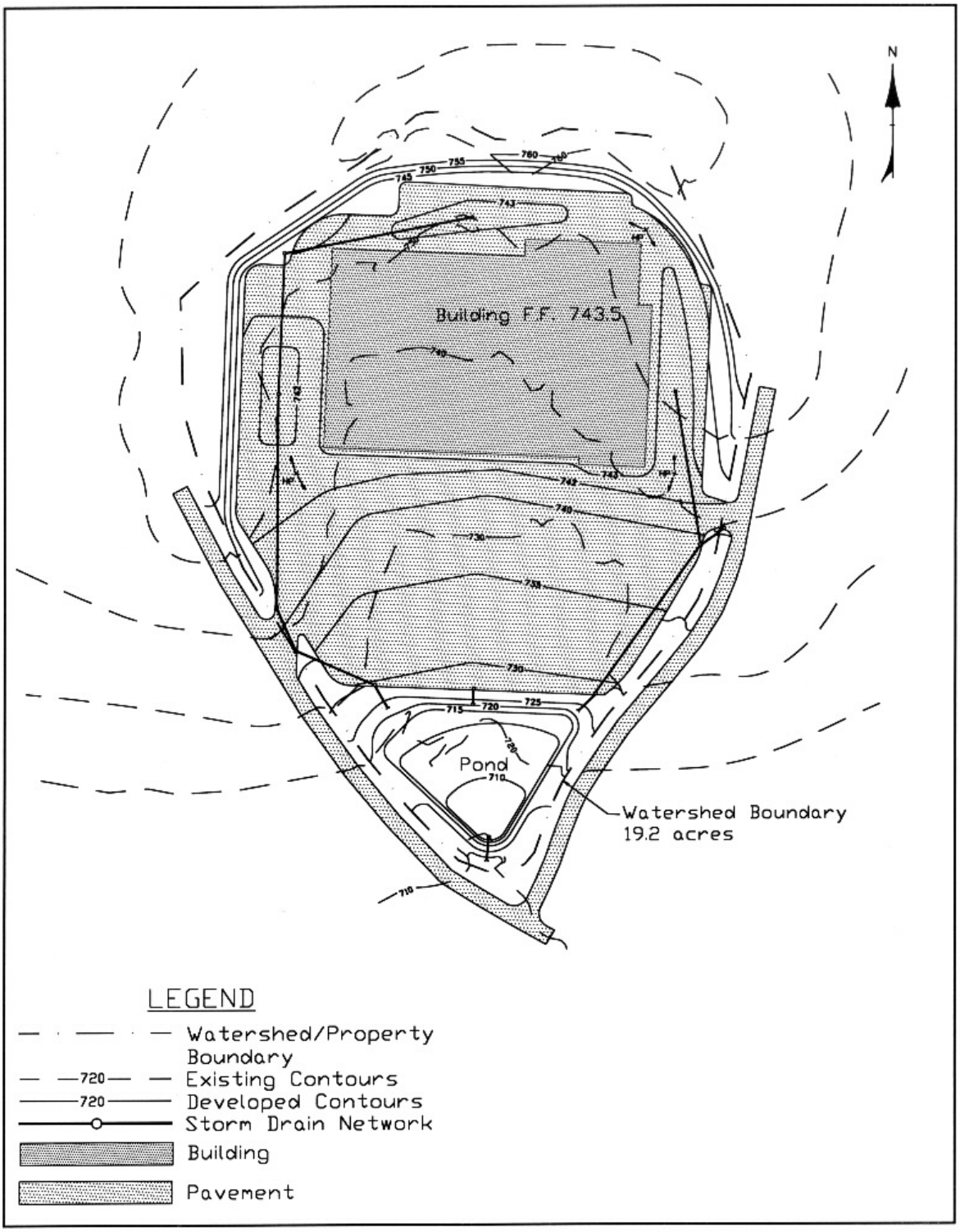

Figure C-2

Example Sketch – Traditional Development (70%IMP)

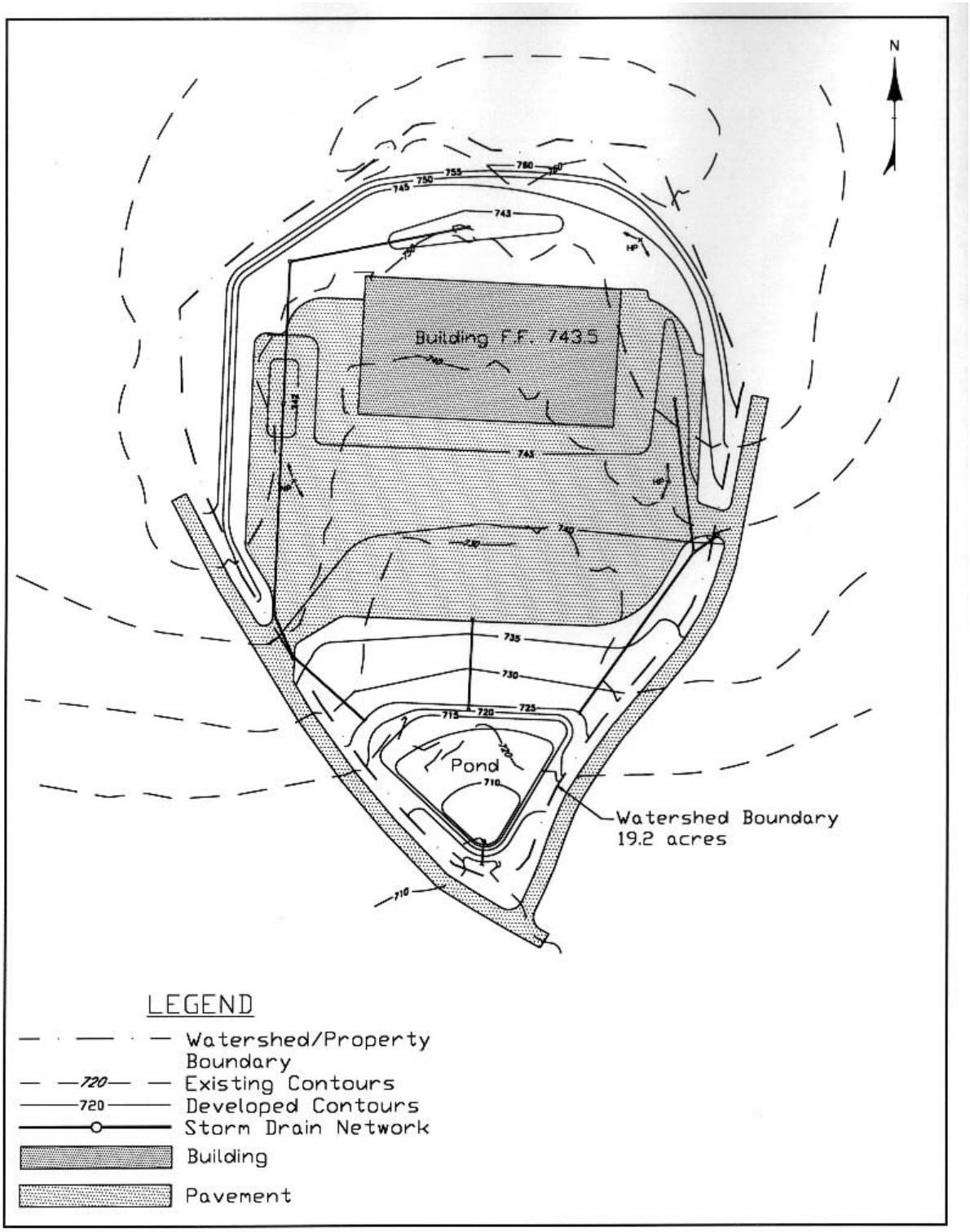

Figure C-3

Example Sketch – Traditional Development (50% IMP)

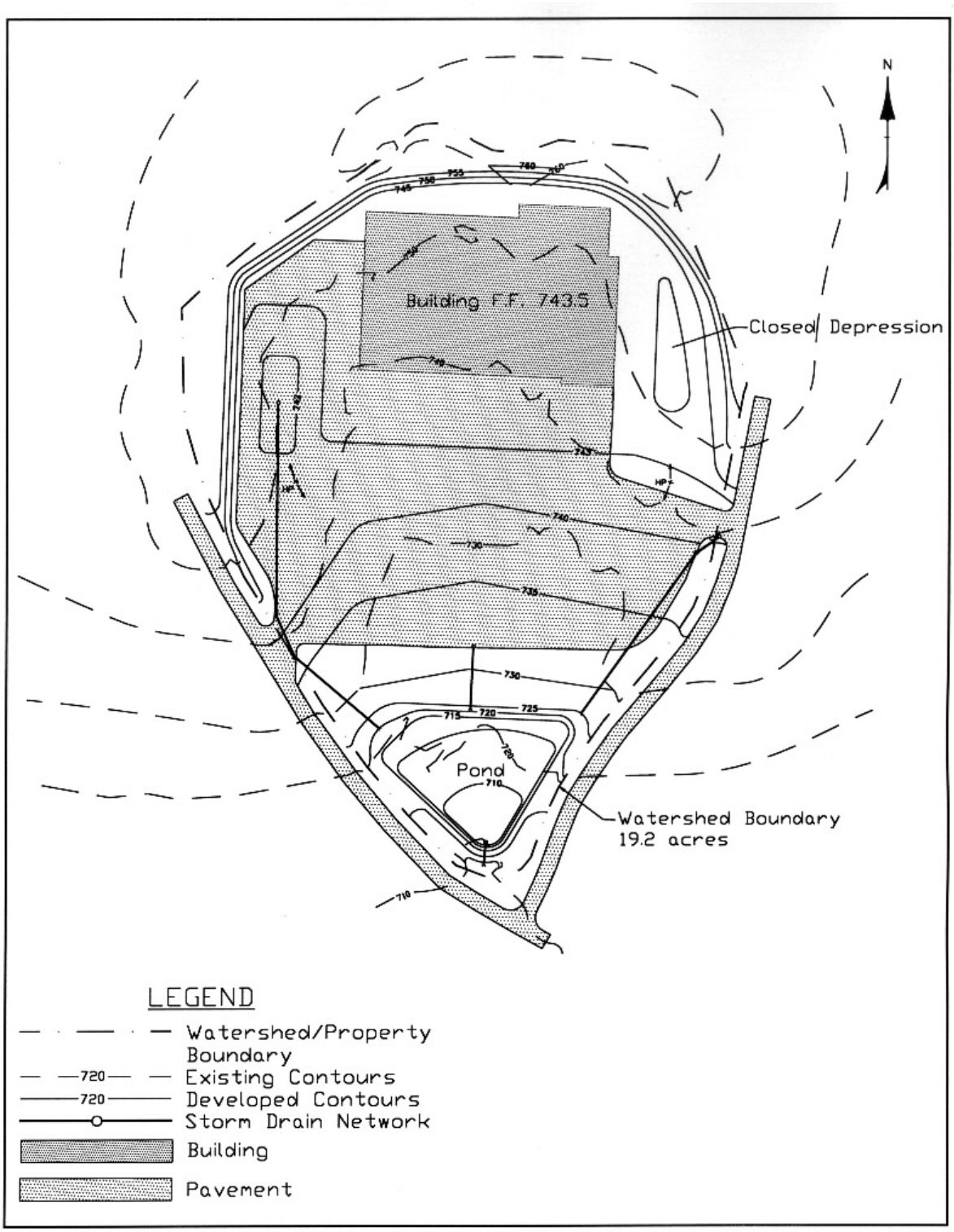

Figure C-4

Example Sketch – Development (60% IMP)
With Minor Recharge/Capture BMP

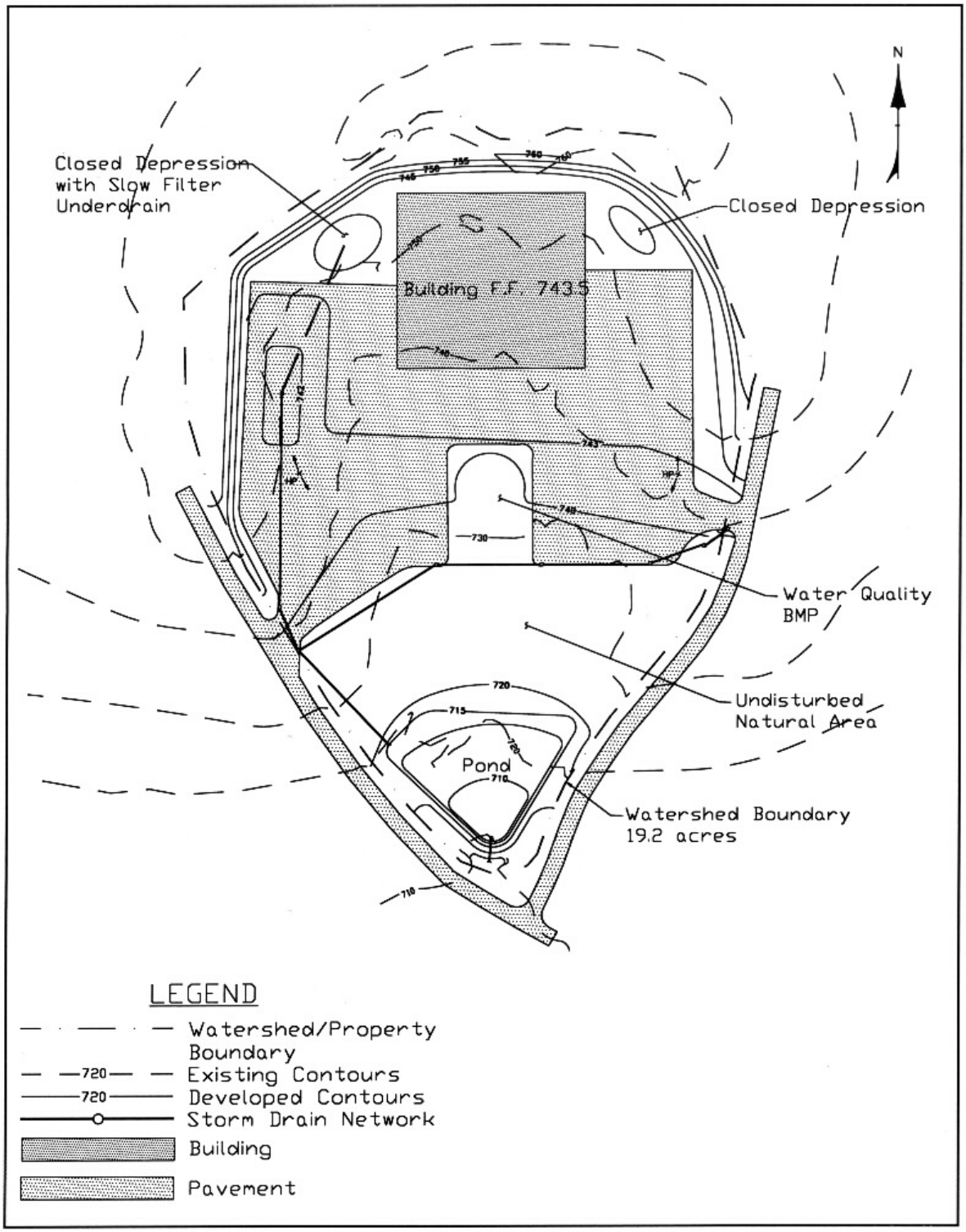

Figure C-5

Example Sketch – Innovative Development (50% IMP)
With Water Quality/Recharge/Capture BMPs and Undisturbed Areas

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TECHNICAL ANALYSIS

INTRODUCTION

The purpose of this Appendix is to describe the analytic processes involved with developing a strategy to manage stormwater runoff from new land development within the Spring Creek Watershed. This analysis considered the following objectives:

1. Develop a strategy for analysis that includes the influence of the karst characteristics of the region on hydrologic response; and
2. Establish design standards which are consistent with natural hydrologic processes to manage stormwater runoff and its impacts on water quality, flooding, channel stability, and groundwater recharge within the watershed.

The first objective is met through detailed evaluation of historic data and validation of sub-watershed runoff models. The second objective is met through the establishment of design standards for managing flooding, reducing channel erosion, maintaining groundwater recharge, and implementing water quality control methodologies.

KARST INFLUENCES ON HYDROLOGIC RESPONSE

The Spring Creek Watershed is almost entirely underlain by carbonate rock formations. The resulting karst nature of the Watershed produces hydrologic characteristics that do not always comply with the underlying assumptions and methodologies inherent in accepted surface water hydrologic models. For this reason, the initial focus of the Stormwater Management Plan was to develop a strategy for analysis that accounts for the hydrologic characteristics resulting from the karst nature of the watershed. To accomplish this, an intensive analysis of historical data was performed including hydrologic simulations of sub-watershed areas.

It is noted that, a full watershed study was not conducted as a part of this study for the purpose of instituting release rate districts. The reason is that the release rate concept of watershed stormwater management is founded solely on the assumption that the unit hydrograph,

infiltration excess, and surface runoff concepts are valid within the basin. Research conducted to date in this study, and previously by others, indicates that, while the underlying assumptions of infiltration excess on a watershed scale are almost never valid anywhere in the northeastern United States, they are particularly not valid in the Spring Creek Basin. In addition, release rate analysis is based on manipulation of sub-area hydrograph timing, which is the most uncertain (and often misunderstood) parameter in any unit-hydrographing model.

Approach

Hydrologic simulations and extensive analysis of historic data were used to substantiate the typical karst influences on hydrologic response within the Spring Creek Watershed. The initial effort focused on validation of a hydrologic model of a small, karst sub-watershed area. The Shiloh Road sub-watershed was selected for several reasons. First, it demonstrates characteristics that are consistent for karst regions throughout the Spring Creek Watershed. Secondly, it is an area that is projected to be highly developed, and will provide an excellent case study for the evaluation of existing and future drainage conditions in a karst subarea. Figure E.1 shows the Shiloh Road watershed in relation to the Spring Creek Basin and Figure E.2A and Figure E.2B show existing conditions and a hypothetical development scenario based on future zoning conditions respectively. Finally, a small sub-watershed was used because it allowed for a more intensive and realistic analysis.

Unfortunately, as with many small watersheds, there are no data available that would permit a true calibration of the Shiloh Road Watershed. Therefore, the goal was to develop a validation approach that would produce results that were reasonable, implementable, and transferable to similar subareas within the Watershed. Once this was done, a hydrologic model was developed for the Shiloh Road Watershed that used the “calibrated” baseline value for comparative purposes. Following this analysis, an actual watershed development scenario was designed and used in the model analysis.

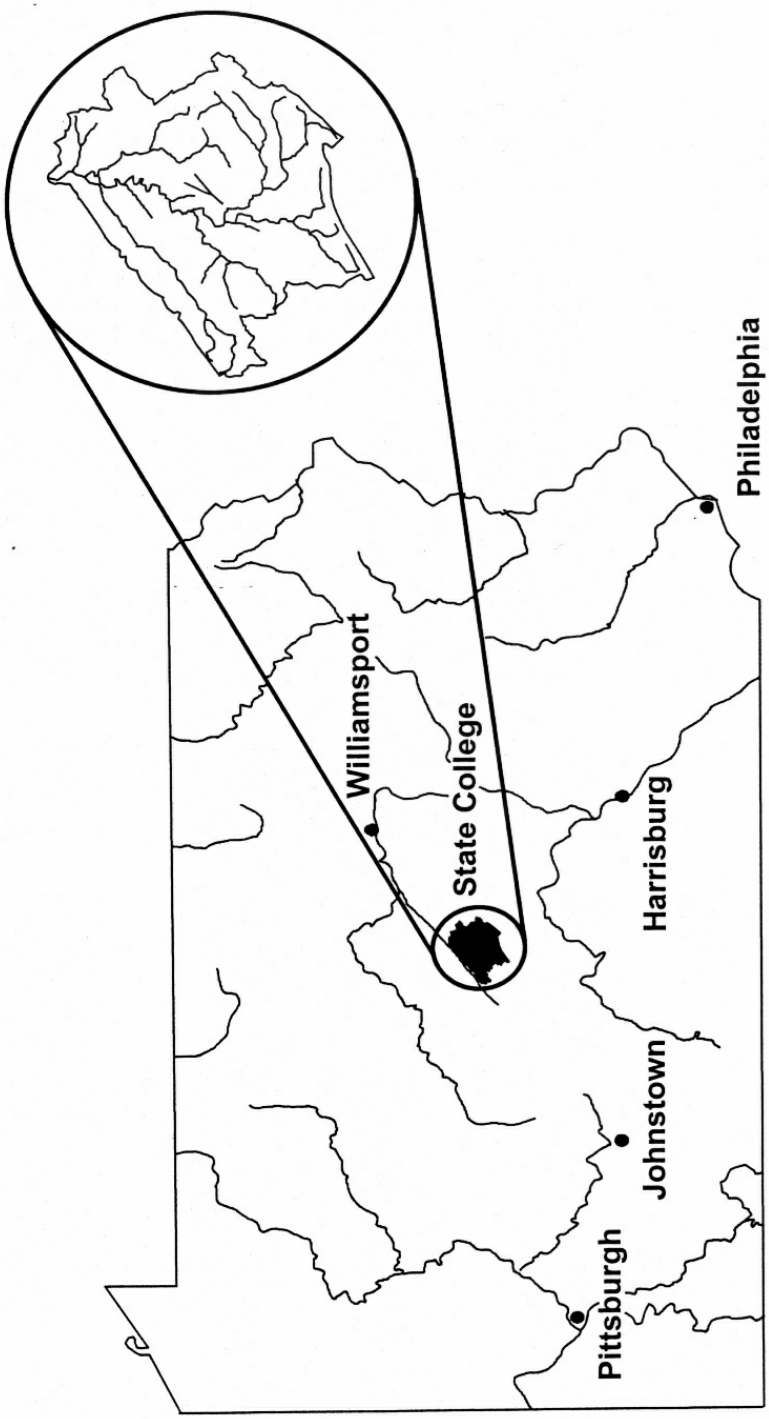


Figure E.1 Location of the Spring Creek Watershed in Pennsylvania

Figure E.2A and E.2B

11X17

Model Selection

Due to its carbonate geology, the selection of a model to best represent the Shiloh Road Watershed at first appears difficult. A process oriented groundwater model that could account for diffuse or conduit types of flow would at first appear to be the model of choice. However, these models generally are difficult to use or run. More importantly, because of the large amounts of data required, none of which were available for the Watershed, default values would have to be used, reducing the model to no better (and perhaps much worse) than a predictive "black-box" surface water model. Once this Stormwater Management Plan was put into effect, engineers and designers would be required to use the methods developed in the analysis. Therefore, a simpler predictive model needed to be selected. Additionally, the selected model needed to adequately incorporate the ability to simulate future development within the Watershed, where a large portion of the hydrologic process is altered to surface runoff. Therefore the decision was made to use a simpler, but well-established, predictive surface water model.

There are numerous predictive models that could have been used for this analysis and later be recommended for use in a model ordinance for the analysis of storm drainage systems and stormwater management purposes. Some of the newer versions of these models have water quality components. Some of the more prominent and recognized models that were reviewed were: the USGS's Distributed Routing Rainfall Runoff Model (DR3M-QUAL); the EPA's supported Hydrologic Simulation Program (HSPF); the Illinois Urban Drainage Area Simulator (ILLUDAS); the Penn State Urban Runoff Model (PSURM); the Storm Water Management Model (SWMM); the Corps of Engineers' Hydrologic Engineering Center's HEC-1; and the NRCS's "Computer Program Project for Project Formulation Hydrology", Technical Release 20 (TR-20).

Although water quality was a secondary goal, those models that had water quality modeling capabilities were not considered because of their complexity and overall data requirements. The TR-20 computer model was selected because of its wide acceptance by users, the ready availability of data and documentation, and because of its ability to be modified somewhat to account for carbonate geology. Each of these reasons is expanded on in the following sections.

In 1995, Hagen published a study on the hydrologic models used for small urban watersheds in the United States. He found that of 20,975 hydrologic studies conducted in the United States, 10,763 (51.3 percent) used the method of the National Resource Conservation Service's (NRCS) Technical Release No. 55 (TR-55). TR-55 is a graphical or tabular approximation of the TR-20 computer model output and is not a computer model itself, although it is available on computers. Additionally, the NRCS's Technical Release No. 20 (TR-20) method accounted for another 9.3 percent for a combined total of over 60 percent (Hagen, 1995). It is clear that the NRCS methodologies are being used for the majority of small stormwater management projects in this country. Although no such study has been conducted specifically for Pennsylvania, in 1998, Fennessey et al. conducted a random analysis of 50 land development and stormwater management ordinances in Pennsylvania. The use of TR-55 was recommended or required in 49 of the 50 ordinances for small watersheds, while the other required TR-20.

The TR-55 and TR-20 methods have not only gained wide acceptance among engineers and designers, but also by regulators due to their ease of use. Unfortunately, Hagen also noted that users of established models and the regulatory personnel who enforce them often resist or adapt very slowly to changes in stormwater management computational methodologies. Hagen pointed out that numerous users were still using the NRCS's 1975 version more than a decade after being replaced by the 1986 version. Additionally, the time and resources required to retrain or educate users and regulators alike if another method were proposed would be substantial. This is one of the reasons the TR-20 model and its approximation technique, TR-55, was selected for use in this study.

The second and most important hydrologic reason the model was selected is because it is the best-documented hydrologic model in the United States and has the largest available database of model parameters for users. Although the TR-20 and TR-55 users manuals are readily available, the NRCS's National Engineering Handbook, Section 4 – Hydrology (NEH-4), which is the documented basis of the TR-20 model structure, is also readily available. Additionally, Hydrologic Soil Group data, which is used for the selection of the curve number (to represent the transformation of rainfall into runoff), is available for any land area in Pennsylvania. The TR-20 model has also been calibrated and studied by numerous independent researchers over the last 40 years.

Finally, the structure of the model, which is essentially based on taking a direct runoff volume and estimating a peak runoff rate through the use of a dimensionless unit hydrograph and a hydrograph timing lag (or conversely the time of concentration), allows one to modify any portion of the model to better represent the Spring Creek Watershed runoff characteristics due to the carbonate geology. Although both the unit hydrograph and the curve number have come under fire in the last two decades, no method has been developed to provide better estimates for any ungaged watershed across the United States. More complex, physically based models or simpler statistical models do not generally allow adjustments to be made to their structure and should not be used in areas that are not representative of the areas for which they were developed.

Nonetheless, points need to be kept in mind regarding the NRCS's runoff models. There have been situations where the TR-20 or TR-55 model has been used improperly or did not represent the watershed conditions accurately. However, when used by trained hydrologists most erroneous watershed estimates can be detected due to experience with watershed data. This reinforces a statement by the NRCS in the TR-55 manual (USDA-SCS, 1986) that states, *" Only through an understanding of these* (watershed) *characteristics and experience in using these models can we make sound judgement on how to alter model parameters to reflect changing watershed condition".* This is also true for making any hydrologic estimate. Unfortunately, the majority of users of TR-55 for small watershed analysis have limited, if any experience with real watershed data and generally lack extensive knowledge of hydrologic processes. Although this fact makes the selection or recommendation of a model for general use difficult, it reinforces the decision not to use a more complex physically based model.

In summary, the NRCS's TR-20 model was selected because of its wide acceptance by users, the ready availability of data and documentation, and because of its ability to be modified somewhat to account for carbonate geology. Another hydrologic model chosen for the analysis would not allow this ability or comfort level in modifying its structure.

Prologue To Calibration

Prior to attempting to calibrate the NRCS's TR-20 Hydrologic Model for the Shiloh Road Watershed, a series of analyses first needed to be conducted. These analyses were required because there is no actual runoff data for the Shiloh Road Watershed. Calibration in this sense implies that the model estimates for the existing watershed conditions needed to be within a reasonable order of magnitude of the actual runoff conditions using the best available data and methods and considering actual hydrologic processes in order to determine what a reasonable value should be. Once it was determined that the model estimates were reasonable, the estimates could then be realistically considered a valid baseline from which other modeling or development scenarios could be tested. Without a realistic baseline, differences in model outputs for different development scenarios would be considered highly questionable. Additionally, to generate a reasonable estimate, changes to the model, would need to represent as closely as possible the actual hydrologic processes that occurred within the Watershed. The goal of the following work, therefore, was to develop modifications to the model that reflected as near as possible the actual hydrologic processes, first from a primarily undeveloped karst watershed and then to an urbanizing watershed.

The first step in the calibration was a matched pair analysis using 16 watersheds (8 pairs) throughout Pennsylvania ranging in size from 1.68 to 87.2 square miles (refer to Figure E.3). each pair consisted of a watershed that had a portion consisting of carbonate geology and a watershed without carbonate geology. Pairs were also selected such that they were both from the same general area so that regional variations, such as precipitation, could be considered minimal. Two of the matched pairs included watersheds located on Spring Creek, with a third matched pair being similar in size to the Shiloh Road Watershed and located within the same geologic region.

In reference to the TR-20 model, the matched pair analysis also showed that the TR-20 model over-estimated the runoff rates for most of the watersheds underlain by carbonate rock when traditional methods of determining input parameters (CN, Tc) were used. This indicated that if the TR-20 computer model were to be used, it would require adjustments to the model structure to account for the carbonate geology.

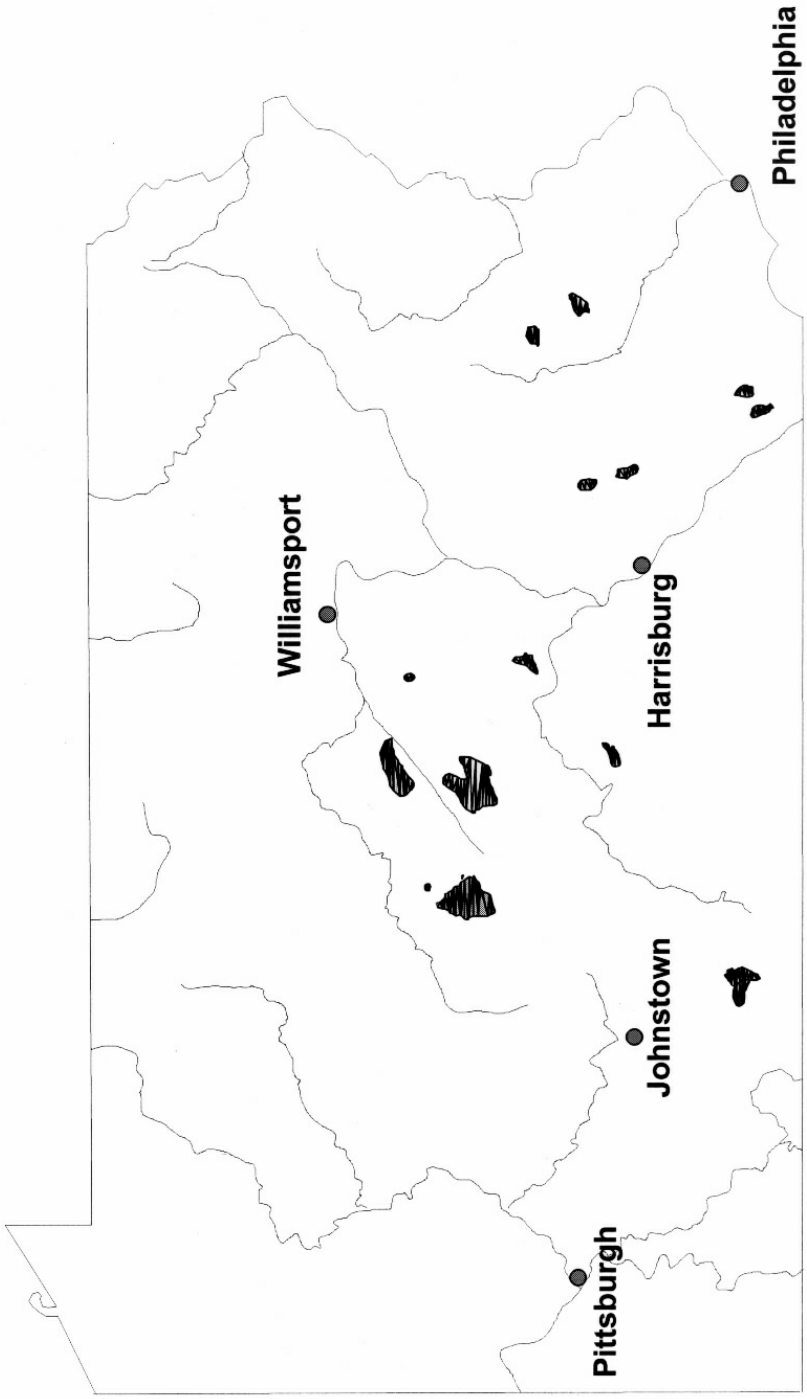


Figure E.3
Location of Matched Pair Watersheds in Pennsylvania

Although our current knowledge of the hydrologic process clearly recognizes deficiencies with unit hydrograph models, it is believed that the TR-20 model could still be reliably used for engineering design purposes with certain adjustments or alterations. There are three basic types of alterations that can be used to modify the NRCS's TR-20 hydrologic model. The first would be to redefine the shape of the dimensionless unit hydrograph used in the model. The second would be to modify the timing methodology used to estimate the watershed lag (or conversely the time of concentration), and the third modification would be in the curve number (CN), which is a rainfall to precipitation transformation parameter. It was decided that prior to altering any of these three, a comprehensive analysis had to be made of actual historical data from the region.

It was decided that the 58.5 square mile watershed to the USGS gage located at Houserville, PA on Spring Creek would be best to use for such an analysis. The Houserville gage was installed in 1985 and has continuous flow data available from the USGS. Continuous hourly precipitation data for the same period was available from the Pennsylvania State University's Walker Building. Additionally, extensive analyses have been conducted within the basin, including one by Taylor (1997) on the water budget for the Spring Creek Basin which included the Houserville gage. Three other USGS gages were located within the basin at Axemann, Milesburg, and Bellefonte from which data were available. The Shiloh Road Watershed outlets on Spring Creek between the Houserville (upstream) and Axemann (downstream) gages.

Using actual data, extensive hydrologic information was extracted for the basin. Some of these data were: the seasonal variability of precipitation and runoff, including the losses or gains between gages and the statistical probability of flood magnitudes for each actual gage. In reference to unit hydrograph methods, additional data was collected that included: the CN Infinity, the hydrologic response, and the annual runoff to precipitation (Q/P) ratio. A unit hydrograph and the watershed lag were also extracted for the Houserville gage. Additionally, several site investigations were conducted during or directly following precipitation events. Each of these analyses is further described in following sections.

Matched Pair Analysis

Prior to attempting to calibrate the NRCS's TR-20 Hydrologic Model for the Shiloh Road Watershed, a series of analyses were conducted using other similar watersheds. First was a

matched pair analysis conducted using 16 watersheds (8 pairs) throughout Pennsylvania ranging in size from 1.68 to 87.2 square miles (see Figure E.3). This was done in order to test for the effect of the carbonate geology on peak runoff rates. The 16 watersheds used in the study were selected specifically so that each pair of watersheds actually qualified as a classical matched pair experiment in which equivalent runoff data were collected for both watersheds over the same period of time. Each pair consisted of a watershed that had a portion consisting of carbonate geology while the other watershed in the pair did not (or contained very little). Pairs were also selected such that they were both from the same general area so that regional variations such as precipitation could be discounted. Two of the matched pairs included watersheds located on Spring Creek (USGS Houserville and Axemann gages), with a third matched pair being similar in size to the Shiloh Road Watershed and located within the same geologic region as Spring Creek (refer to Table E.1). The base data used in the matched pair analysis can be found in an unpublished report by Sweetland Engineering & Associates, Inc.

General watershed data for 12 of the 16 watersheds were taken from the appendix of PSU-IV, while the remaining four had the data reduced by the same methods that were originally used in determining the PSU-IV database. While some would argue that using watersheds that were used in the development of PSU-IV would not allow for the testing of differences in the estimation of PSU-IV and another model, the parameters used in PSU-IV model (Sy and G) for each watershed estimate were taken from the generalized maps, and therefore, the model was being tested in some fashion.

Once the watersheds were selected, the analysis was conducted in the following manner.

1) An historical analysis was conducted of the annual maximum peak runoff rates using the USGS gage data with the Miniex computer program to determine the log Pearson III probability distributions for the 1-, 2-, 5-, 10-, 25-, 50-, and 100-year return periods.

2) The Excel program was used to plot log normal plots of the historical data and the miniex output for each probability distribution using a Weibul plotting position. Although this plot does not always provide the best linear relationship, it does allow the visual determination of how well the computed distribution matches the

Table E.1
Watersheds Used for the Matched Pair Analysis

Pair ID	Gage #	Name	Lat	Long	Years of Record From	Years of Record To	Sq. Miles	County	Basin
p1	1557100	Schell Run at Tyrone	404000	781500	57	81	1.68	Blair	Upper Juniata
k1	1548020	Bull Run Near Loganton	410030	771935	63	81	1.99	Clinton	Bald Eagle
p2	1565920	Lick Run Near East Waterford	402115	773855	62	81	8.38	Juniata	Lower Juniata
k2	1565700	Little Lost Creek at Oakland Mills	403619	771842	60	81	6.52	Juniata	Lower Juniata
p3	1578200	Conowingo Creek Near Buck	395035	761145	63	98	8.71	Lancaster	Lower Susquehanna
k3	1578400	Bowery Run Near Quarryville	395341	760650	63	81	5.98	Lancaster	Lower Susquehanna
p4	1572900	Reeds Creek Near Ono	402425	763315	62	81	8.63	Lebanon	Lower Susquehanna
k4	1573086	Beck Creek Near Cleona	401924	762900	64	80	7.87	Lebanon	Lower Susquehanna-Swatara
p5	1600700	Little Wills Creek at Bard	395535	783940	61	81	10.2	Bedford	North Branch Potomac
k5	1559700	Sulphur Springs Creek Near Manns Choice	395840	783708	62	78	5.28	Bedford	Raystown
p6	1470720	Maiden Creek Tributary at Lenhartsville	403423	755234	62	79	7.46	Berks	Schuylkill
k6	1471800	Pine Creek Near Manatawny	402443	754402	61	80	9.7	Berks	Schuylkill
p7	1547700	Marsh Creek at Blanchard	410334	773622	56	98	44.1	Centre	Bald Eagle
k7	1546400	Spring Creek at Houserville	405001	774940	84	98	58.5	Centre	Bald Eagle
p8	1542000	Moshannon Creek at Osceola Mills	405058	781605	36	93	68.8	Clearfield	Upper West Branch Susquehanna
k8	1546500	Spring Creek Near Axemann	405323	774740	36	98	87.2	Centre	Bald Eagle

historical data. It was determined that for all of the watersheds, the Log Pearson Type III distributions visually fit the data best up to a 10-year return period. Therefore, the Log Pearson Type III distribution was used to define the historical maximum peak runoff rate for the 2-, 5-, and 10 year return periods.

3) The peak runoff rates for the watersheds were estimated using TR-20 and PSU-IV, without carbonate adjustments. Because the historical data represented the annual series, the TP-40, 24-hour precipitation depths were annualized. The annual to partial series divergence is not considered constant for a 1-year return period; therefore, the 1-year estimates were not made using the TR-20 model.

4) The two models were compared using standard error and bias for the best fit probability distribution and the synthetic estimates using the 2-, 5-, and 10-year return periods.

5) The standard error and bias for each watershed was compared in the pair versus the other. The runoff data was reduced to inches per hour to remove minor differences in the watershed areas.

6) A determination was made as to whether the location of carbonate geology in watershed accounted for results.

Although it was not the intent of this study to revise PSU-IV's carbonate adjustment methodology, it was desired to determine if the carbonate adjustments were generally valid using different statistical methodologies. From the matched pair analysis, it was discovered that the PSU-IV method for carbonate watersheds was not adequate across the State. This inadequacy has been previously expressed in personal communiqués by Dr. Gert Aron, the senior developer of the PSU-IV methodology.

One reason the method is inadequate is because PSU-IV assumes that watersheds with carbonate geology have a decay function compared to equivalent watersheds without carbonate geology. However, this is not necessarily always a valid assumption; because, as the matched pair analysis showed, watersheds with carbonate geology can have higher than expected peak

runoff rates, which are caused due to the potential for large amounts of conduit flow rather than diffuse flow within the substrata. Two of the eight matched pairs exhibited this characteristic. Nonetheless, the two watersheds on Spring Creek and the one similar to Shiloh Road did exhibit a consistent decay (Standard Difference = 45.6%, 72.0%, and 54.6%) that could be used in modeling applications. The matched pair analysis suggested that both TR-20 and PSU-IV over-estimated runoff rates as the difference in carbonate rock increased within the pair (refer to Figure E.4). Further analysis of this trend is beyond the scope of work of this project.

The matched pair analysis also showed that PSU-IV peak runoff rate estimates, without carbonate adjustments, provided much more realistic estimates than the TR-20 model when TR-20 was used with traditional methods of determining input parameters (CN, Tc). This indicated that if the TR-20 computer model was to be used, it would require adjustments to the model structure to account for the carbonate geology. (Refer to Figure E.5, which showed the historical data for Pair 1 versus the TR-20 and PSU-IV estimates.)

Because Pair 1 (same size and in the same regional area as the Shiloh Road Watershed) exhibited similar characteristics to the two gaged watersheds along Spring Creek, the pair was further analyzed. Using Pair 1, it was concluded that the TR-20 model over-estimated the historical runoff; while PSU-IV, after adjusting for carbonate geology, under-estimated the historical runoff. After studying the comparisons, it was felt that the PSU-IV model, without any adjustments, provided the best estimates and that this method could also be considered for the Shiloh Road Watershed. Additionally, it was assumed that although the Shiloh Road Watershed had much less of a percentage of forests than the Pair 1 watersheds, the lack of urban land in all of the watersheds allowed for their comparison.

However, prior to making any adjustments or alterations to the TR-20 model to make its estimates more in line with the PSU-IV estimates, a comprehensive analysis of the Spring Creek Watershed with actual runoff and precipitation data was calculated. This analysis was required to validate the general findings of the matched pair analysis.

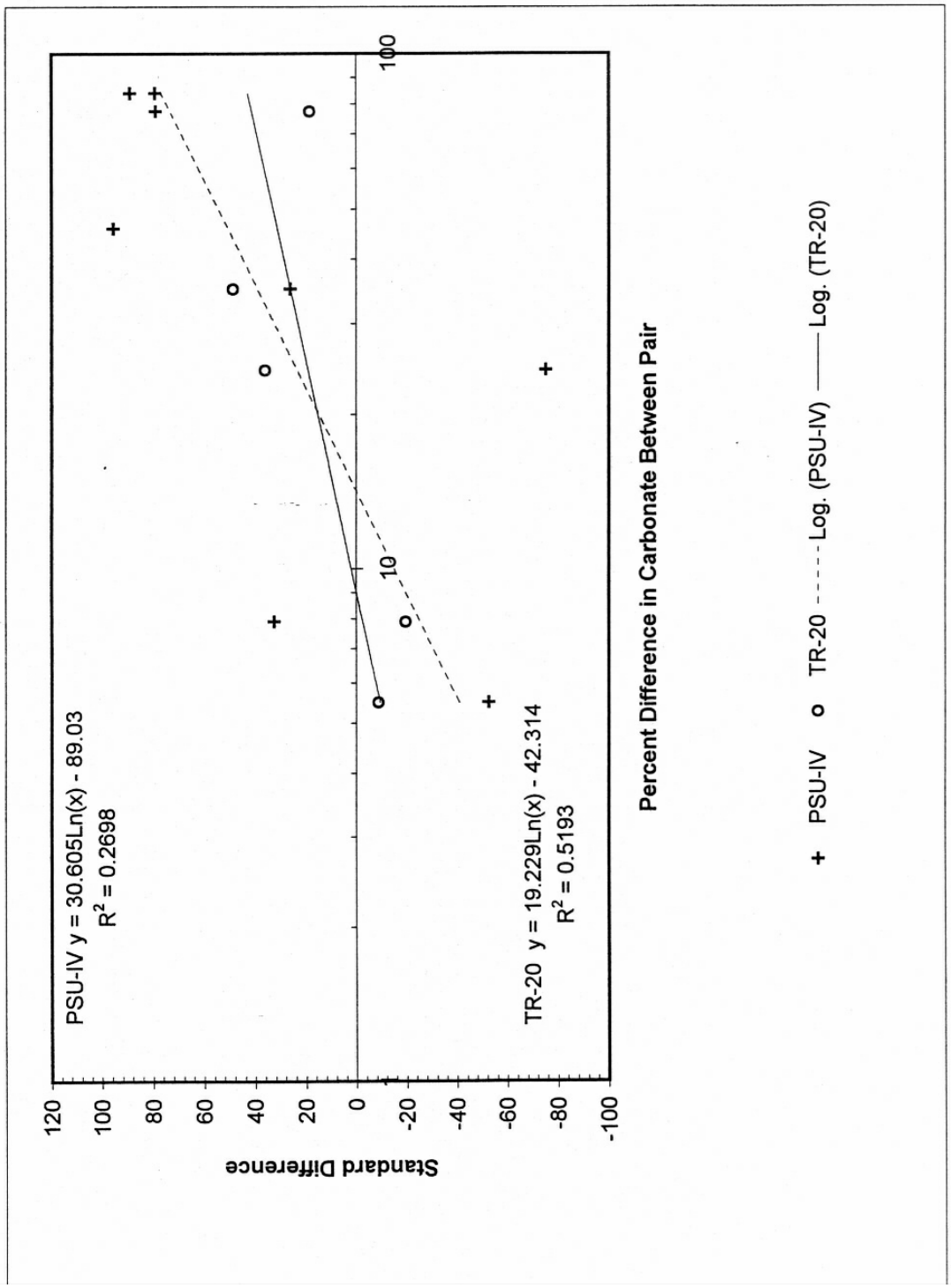


Figure E.4
Relationship of Standard Difference Versus Percent Difference in Carbonate Geology Between Pairs

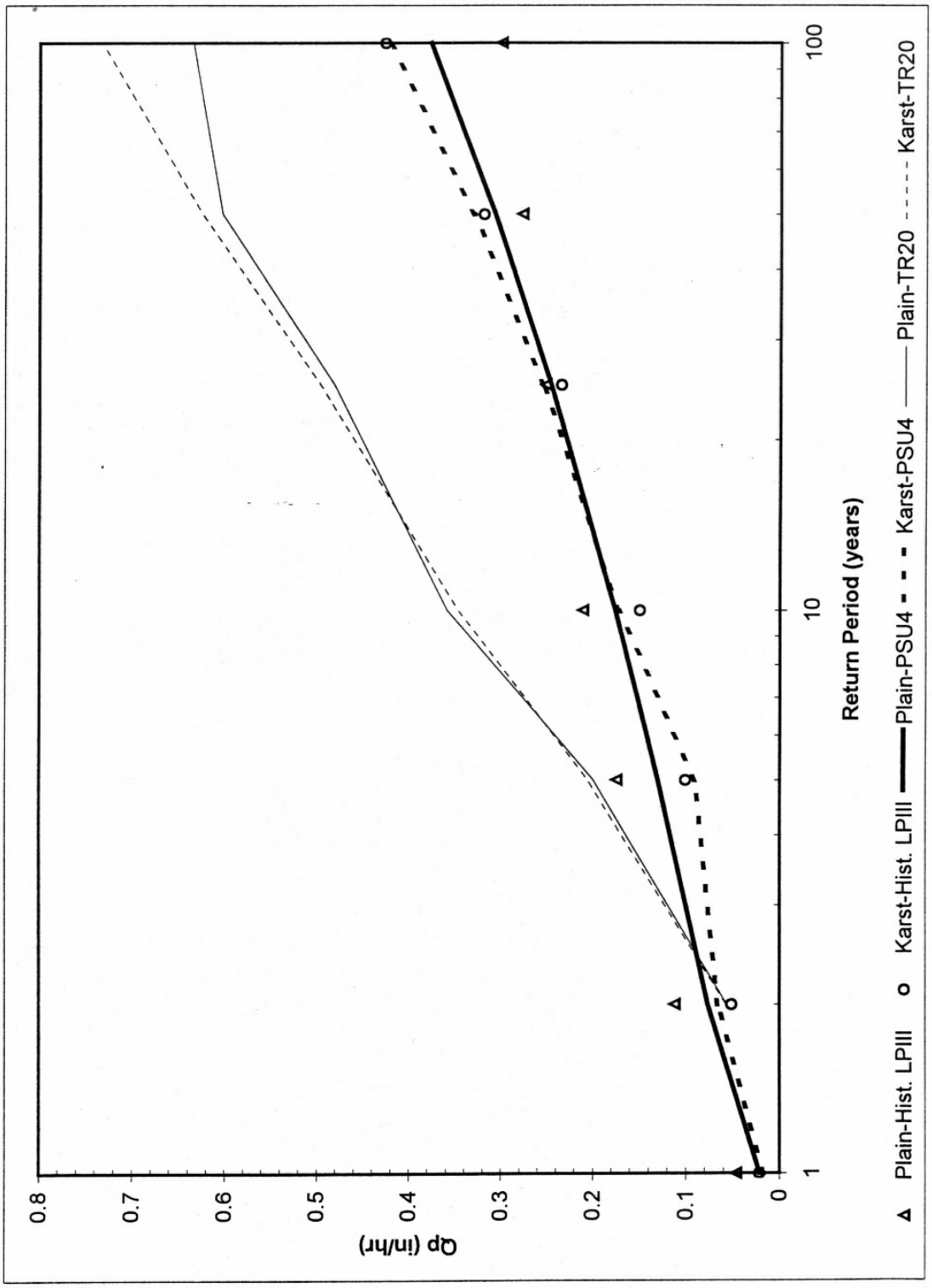


Figure E.5
Example of Matched Pair Results for Pair 1

Seasonal Variations

An analysis was conducted to determine if there were seasonal variations in both the precipitation and runoff that occurred in the Spring Creek Drainage Basin. Daily precipitation values from State College were used for a 10-year period from 1986 to 1996. Precipitation followed a seasonal cycle where the majority of large events occurred during the fall with some larger events also occurring in March (refer to Figure E.6). This was also found in the Weather Bureau's Technical Paper No. 40 (U.S.D.C., 1961) study that produced TP-40. Refer to Figure E.7 from TP-40, which shows the seasonal probability of having an intense 6- or 24-hour rainfall event in the region. Additionally, the previous work by Taylor (1997) on the validity of using only State College precipitation data for the entire drainage basin was reviewed. Taylor found using the method of double mass analysis that precipitation could be considered to act uniformly over the basin as represented by one gage (refer to Figure E.8). This would only be valid at a statistical level, which is acceptable for this study because we are not concerned with directly quantifying cause and effect relationships. These seasonal variations would have a dramatic effect on the runoff from the region because a large majority of the major runoff events are caused by snowmelt events, and for a watershed such as the Shiloh Road Watershed, snowmelt events may be the only time surface runoff reaches the watershed outlet.

Seasonal variations in runoff were checked for the four gage locations along Spring Creek. From smallest to largest, the watersheds were: Houserville, Axemann, Bellefonte, and Milesburg. The major runoff events were also found to follow seasonal cycles. However, in all of the watersheds, the major runoff events occurred in the winter and early spring (refer to Figures E-9 through E-12). From the four graphs, it can be seen that four major runoff events stick out: the 1912 flood, the 1936 flood, Hurricane Agnus in 1972, and the 1996 snowmelt runoff event. The difference in the seasons in which the maximum runoff and precipitation events occurred is typical and would reasonably be expected for a humid climate of the northeastern United States. The data also do not appear to show any distinct trends or effects related to the carbonate geology.

A final analysis was conducted using the Houserville (58.5 square miles), Axemann (87.2 square miles), and Milesburg (145.3 square miles) gages for the period of 1985 to 1998 when each of the three gages was in operation. The runoff peak data (over the baseflow levels

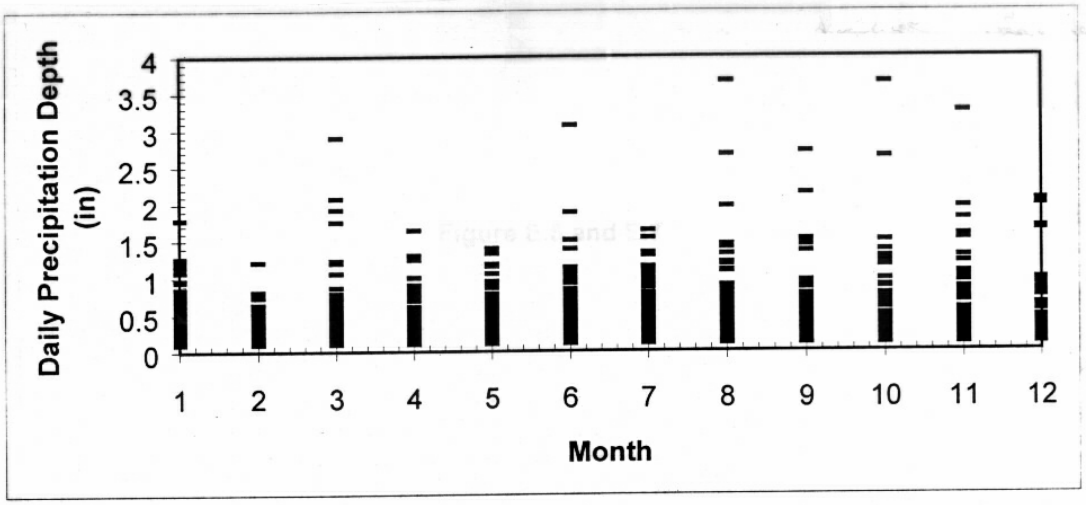


Figure E.6
Seasonal Precipitation Variations for the Spring Creek Watershed

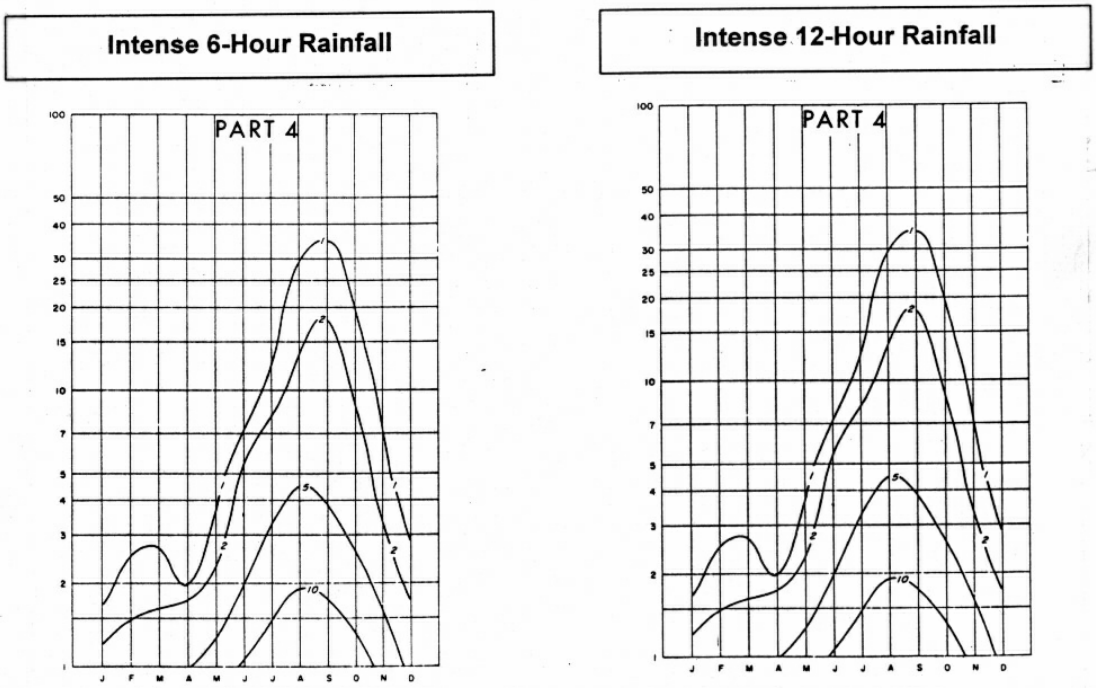
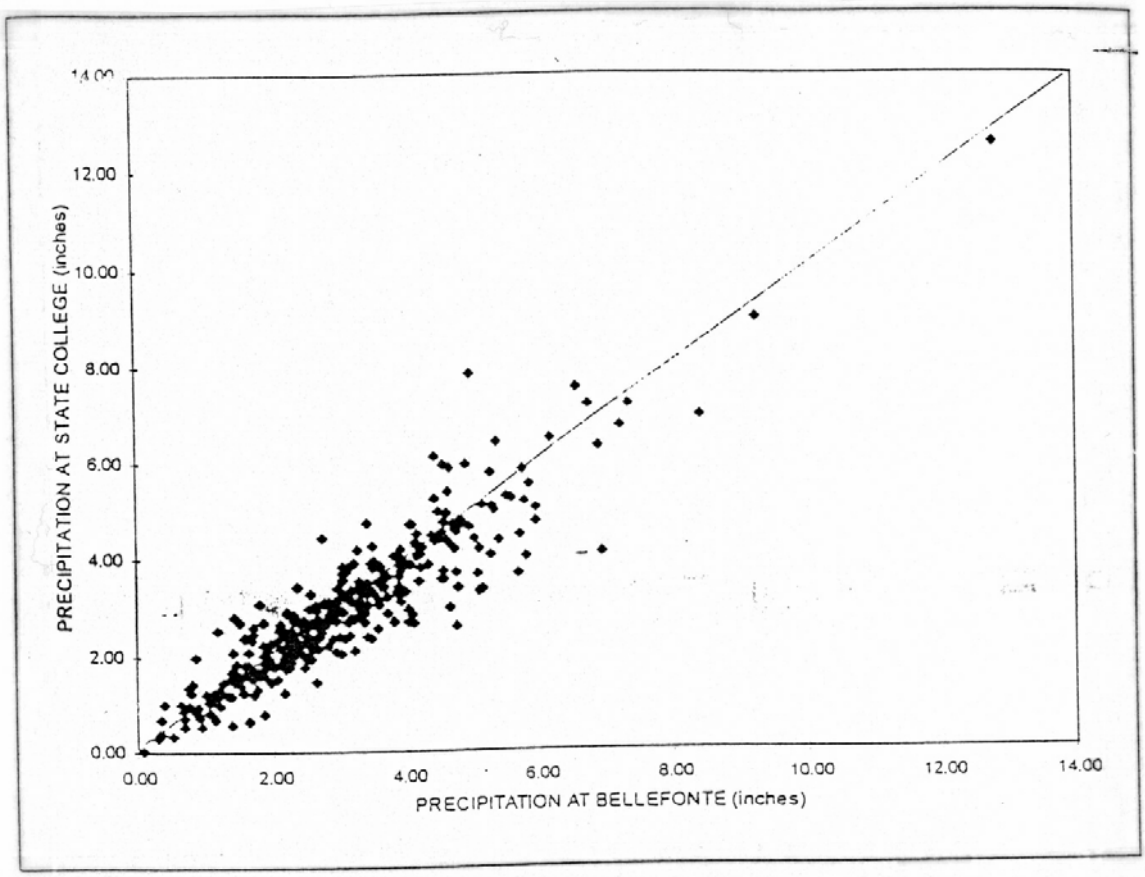


Figure E.7
Seasonal Probability of Intense Rainfall Events



Double Mass Curve of Precipitation at State College and Bellefonte

Figure E.8

Double Mass Curve of Precipitation at State College and Bellefonte

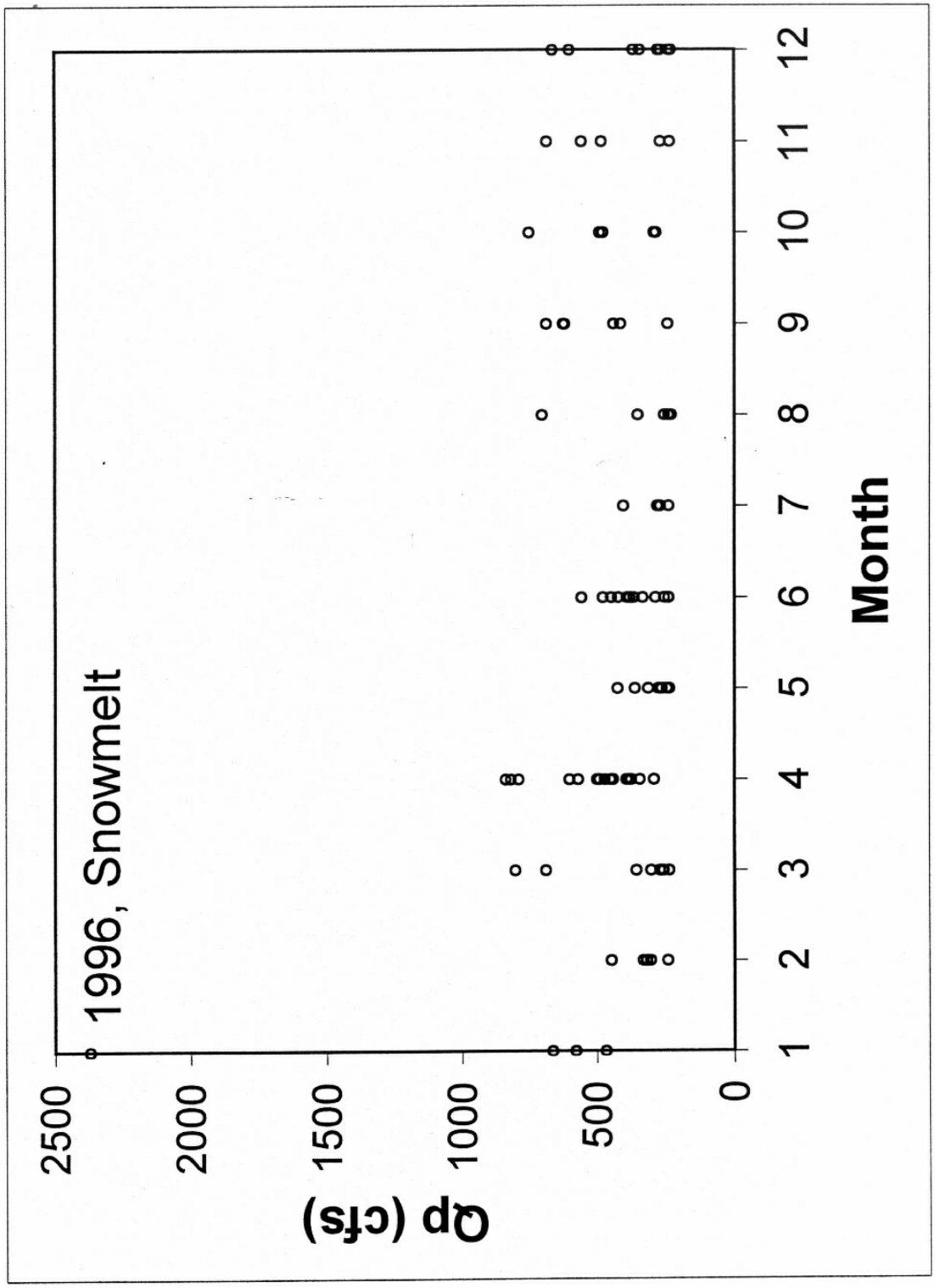


Figure E.9
Annual Series Runoff Events for the Houserville Gage (01546400)

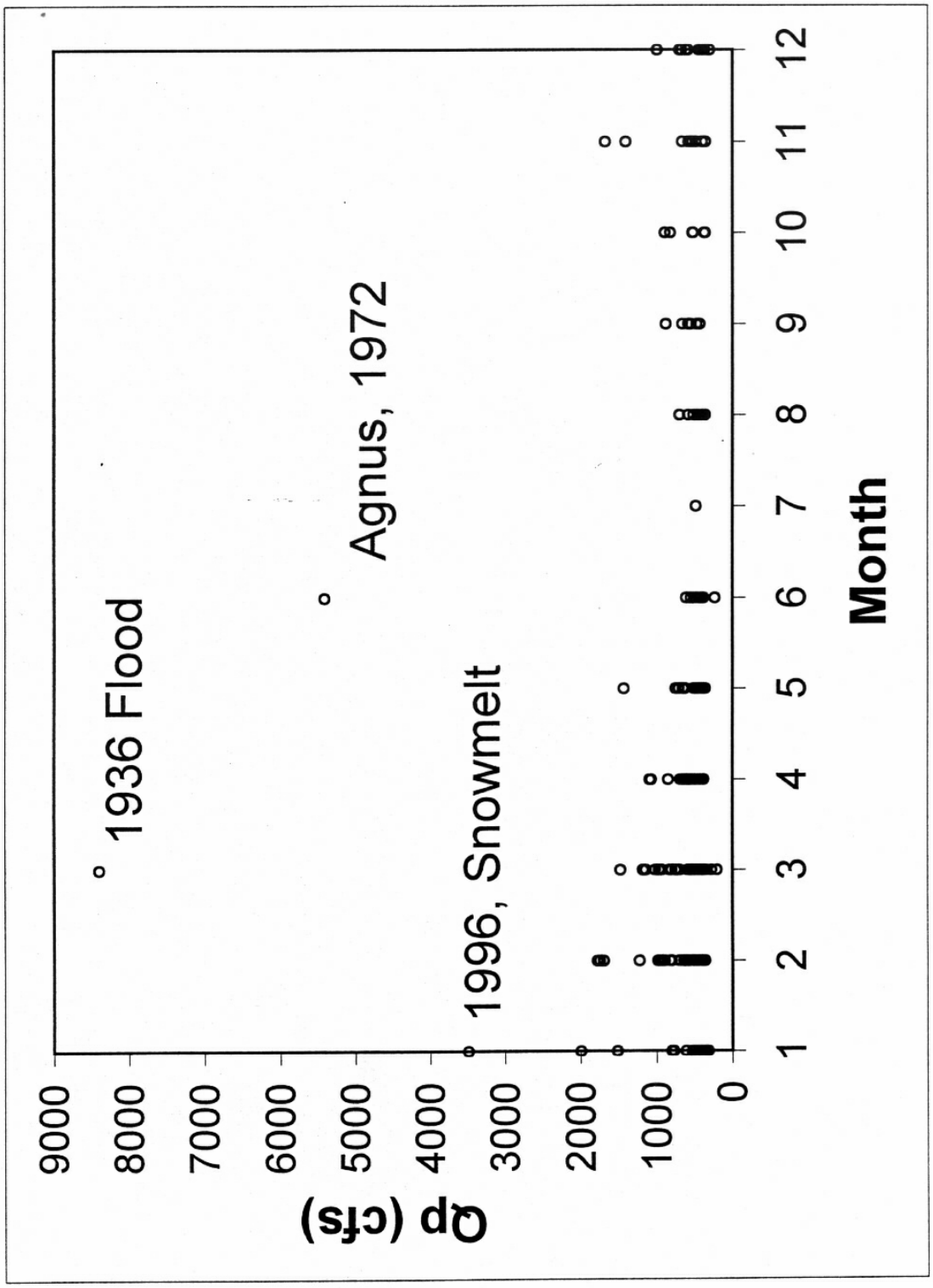


Figure E.10
Annual Series Runoff Events for the Axemann Gage (01547100)

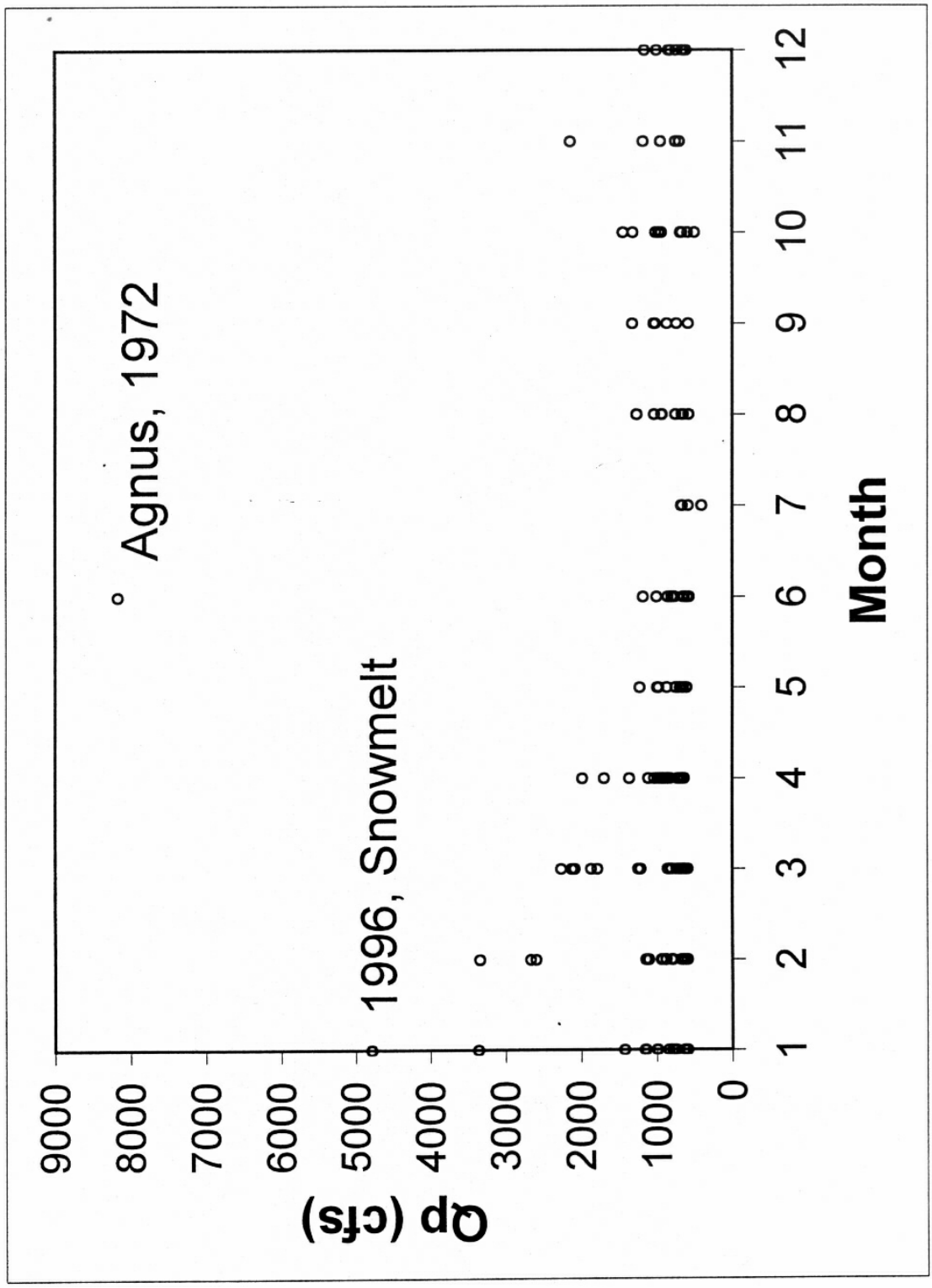


Figure E.11
Annual Series Runoff Events for the Milesburg Gage (01547100)

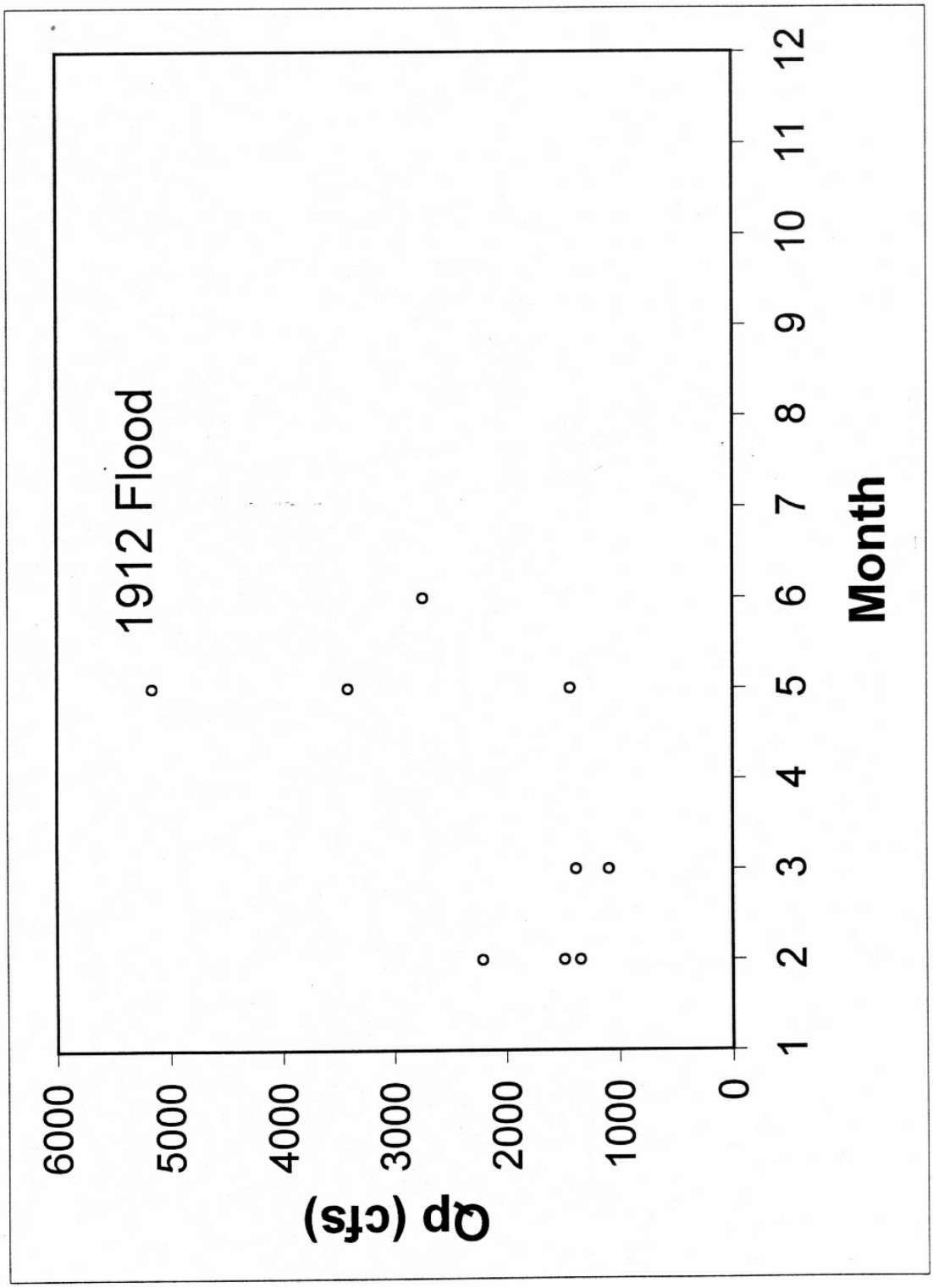


Figure E.12
Annual Series Runoff Events for the Bellefonte Gage (01547100)

provided by the USGS) from the three gages were filtered to find those events where an idealized plug peak of runoff could be observed moving down from the Houserville gage, through the Axemann gage, and then to the Milesburg gage. This was done to validate our previous conclusion that the initial Shiloh Road TR-20 peak runoff rate estimates were too high when compared to the historical runoff rates for the Houserville and Axemann gages and that runoff could not be reasonably expected to decay significantly once they reached Spring Creek (refer to Figure E.13).

From Figure E.14, it can be seen that runoff increases from Houserville to Milesburg, somewhat proportional to the differences in watershed areas, during the major snowmelt events and the spring runoff events. The carbonate geology influence is believed to be minor for these spring or major runoff events. Referring to Figure E.14, it can be seen that the less than extreme events have very similar peak runoff rates at the Houserville and Axemann gages, and in some cases, the storm peaks at Houserville are larger than those at Axemann. This type of decay does not happen at the Milesburg gage. Taylor (1997) noted that the ground water basin to the Milesburg gage is approximately 24 miles larger than the surface water basin, and this may be influencing the difference.

There are five explanations that can explain the trend between the Axemann and Houserville gages. The first is that the precipitation may be higher for some events over the Houserville Watershed. This could be a logical explanation for convectional storms in the fall. However, it cannot realistically account for all the events that show this trend. Another reason this trend is sometimes seen in data is because of runoff storage in the stream overbanks where lateral flow is less than the storage difference. However, the section of Spring Creek between the Houserville gage and the Axemann gage is a narrow valley section that does not have large amounts of overbank storage. The third reason this anomaly shows up in the data could be from gage errors. Because these are well-monitored USGS gages, this is not believed to be the case. The forth reason may be due to underflow at the gages. However, Taylor (1997) concluded that this was most likely not the case. The fifth and most reasonable explanation is that the peaks are decaying as they move between the two gages because Spring Creek is acting as an influent channel. In other words, the stream is losing surface water to the ground water. This action is known to commonly occur in carbonate geologies. Nonetheless, this additional analysis supported that for major runoff events, such as those modeled in TR-20, the

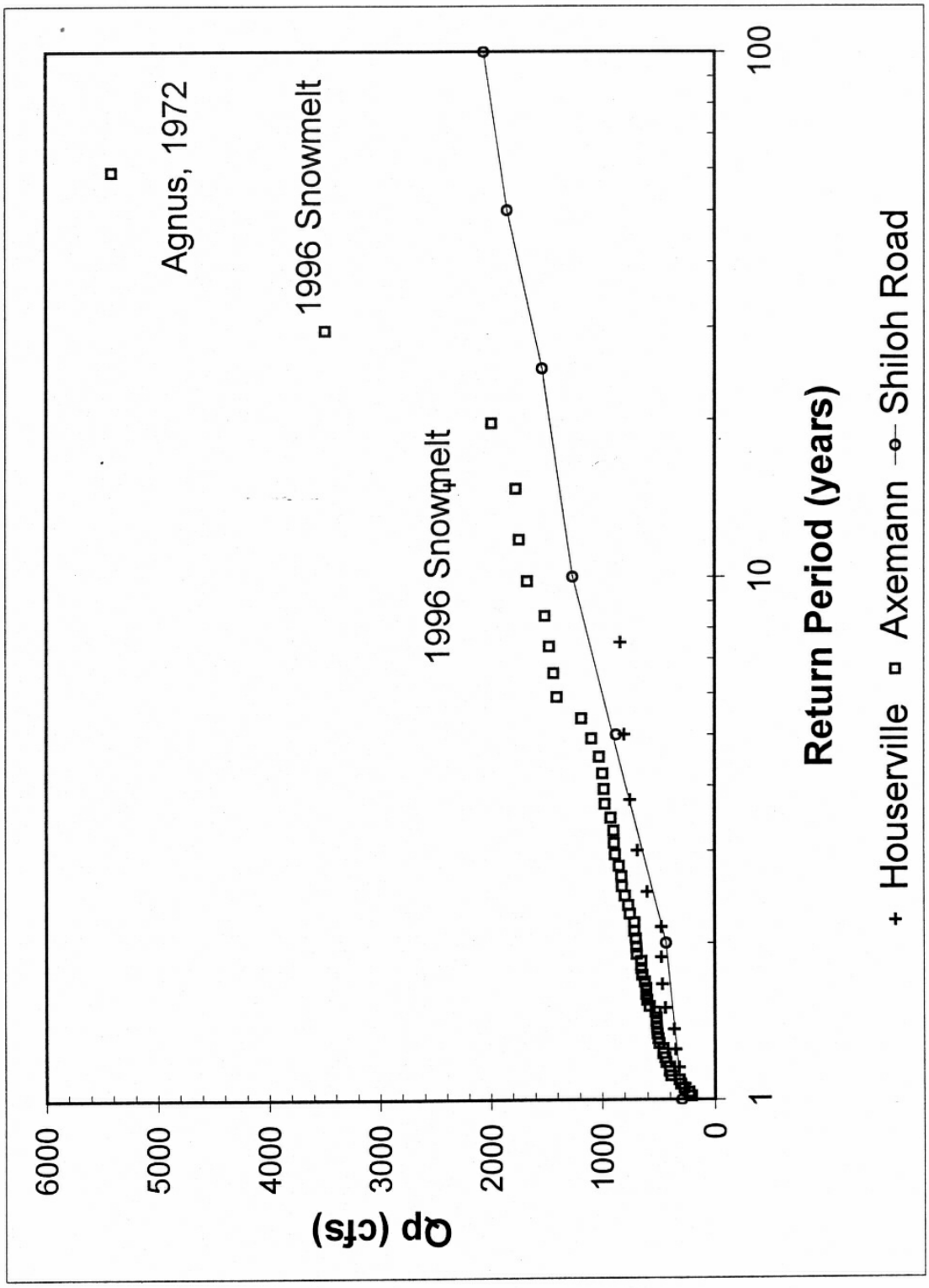


Figure E.13
Comparison of Shiloh Road TR-20 Estimates Versus Historical Annual Series Runoff for Houserville and Axemann Gages

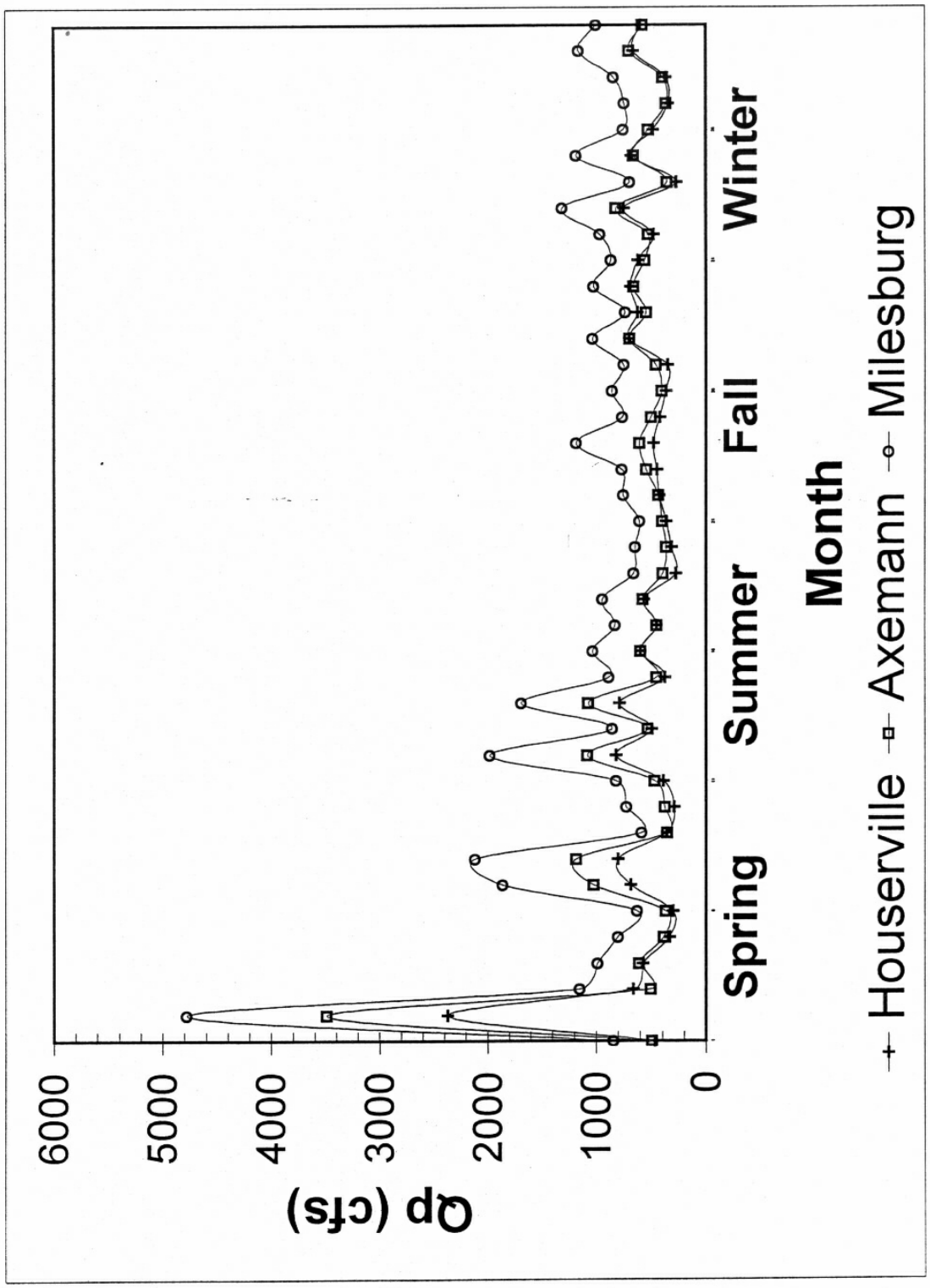


Figure E.14
Comparison of Storm Events as they Move Downstream from the Houserville Gage to the Milesburg Gage

decay required to bring the initial Shiloh Road Watershed estimates in line with the Houserville gage would be unreasonable to expect.

Finally, 1996 had some extraordinary hydrologic conditions beyond having the third highest recorded runoff rate since the early 1920s. A total annual precipitation depth of approximately 58 inches occurred during 1996, almost 20 inches more than average (average at State College is 37.78 inches from 1941 through 1994) and higher than any single year since 1941. This resulted in 16 events being recorded over the baseflow level at the Houserville gage. In 14 years of data collection at the gage, this number of runoff events is higher than any other year on record (next highest is 10, with the average at 5.3). Therefore, the year of 1996 was selected for use in extracting additional hydrologic parameters for the Houserville gage.

Ratio of Runoff to Precipitation

The Q/P ratio was computed for the USGS gage at Houserville located on Spring Creek. It was discovered that the Q/P ratio was 0.39 for both methods for the period of 1986 through 1996 (standard deviation is 0.045, with the range being 0.34 to 0.47). Using Taylor's (1997) data, annual Q/P ratios of 0.4, 0.38, and 0.50 were computed for the Houserville, Axemann, and Milesburg gages respectively. The Milesburg gage is higher because the groundwater drainage area is 24 square miles larger than the topographic drainage area from which P was computed. The Q/P ratios, although slightly higher than would be expected for a non-karst watershed, are within a reasonable order of magnitude for a karst watershed. A generally accepted stream flow percentage using a total annual water balance for the United States is approximately 27% (Hewlett, 1982).

The annual runoff to precipitation ratio (Q/P) was calculated by two methods using a water year from October 1 to September 31. The first method used to compute the Q/P ratio was simply taking the sum of the annual Qs for the full record divided by the sum of the annual Ps for the full record. The second method was to compute the average of ratios of Q/P computed for each year. It was discovered that both methods produced comparable results and did not change the results of the matched watershed pair rank analysis. The second method was therefore selected to represent the data and from here on is referred to as the average annual Q/P ratio.

Additionally, the mean hydrologic response (R) was computed for the Houserville gage for the period of January 1996 to September 1996. The hydrologic response is considered to represent the "flashiness" of the stormflow following rainstorms (Hewlett, 1982). The mean R was found to be 0.079 for rainfall events greater than 1 inch in depth. This value concurs with previous findings by others (refer to Figure E.15). The range of the mean R value for the area represented on the figure is 0.04 to 0.1. From previous work conducted by Hewlett, it was concluded that the mean hydrologic response is controlled more by the geology than by the land use. This appears to be the case for the carbonate geology.

Taylor's 1997 and Gidding's 1974 studies found that the average baseflow for all of the gages along Spring Creek were greater than 80% of the total annual discharges. Although these values are high, it supports the notion that the carbonate geology is resulting in high amounts of delayed interflow with very little surface runoff. Additionally, computing an annual based hydrologic response value from Taylor's data results in a value of 0.07.

In summary, it appears that while the total Q/P ratio is slightly higher than expected, the mean hydrologic response is low. This appears to indicate that the stormflow response is dampened by the carbonate geology.

Curve Number

The curve number (CN) developed by the National Resource Conservation Service (NRCS), formerly the Soil Conservation Service (SCS), is a quasi-empirical parameter that was developed to represent the transformation of rainfall into the storm flow component of runoff (direct runoff). This transformation not only represented surface runoff, but also consisted of unknown proportions of the many forms of interflow. For simplicity of use, prior to the advent of personal computers and at a time when most of the users were still NRCS field technicians, the NRCS choose to vary the CN by only a function of land use, antecedent runoff condition [(ARC, formerly Antecedent Moisture Condition (AMC)] and the Hydrologic Soil Group (HSG) classification. Today the CN is know to be a much more complex function and is known to vary by total precipitation depth, season, land use, ARC, HSG, and the location of the water table. Additionally, the CN may vary due to watershed area, watershed inflection angle, erosion class, geology, land slope, regions, the most influential runoff process, watershed location, and the proximity of soil textures and/or

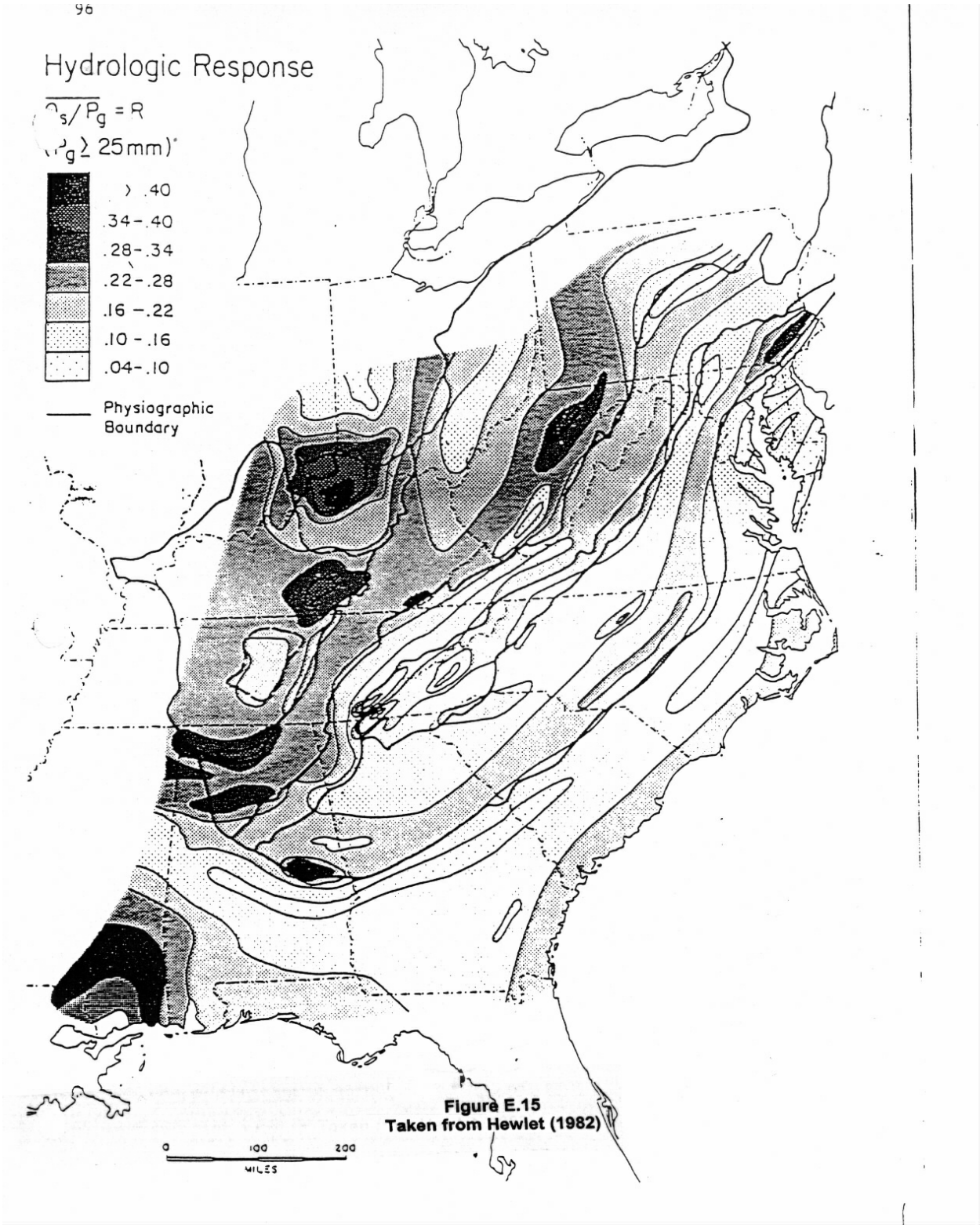


Figure E.15
Taken from Hewlet (1982)

hydrologic soil groups to a drainage way. However, limited practical research or data has emerged in the last 30 years in regards to the CN that allows the average user to either modify CN values beyond the NRCS's table values, or feel comfortable in their estimation.

To use the soil complex methods developed by the NRCA, CNs must be determined for the site or watershed. The standard method used by engineers and designers for assessing CN's, is to select a value from the NRCS table discussed above as a function of land use. Practicing engineers put complete faith in the CN tables published by the NRCS. However, research has shown that simple application of reported CNs in many cases can, produce questionable results. Truly accurate results can only be obtained through a calibration process for CN's.

Additionally, the credibility given to the CN tables is not justified when one understands how the curve numbers were developed. The NRCS's National Engineering Handbook, Section 4 – Hydrology (NEH-4), stated that the literature was searched for watersheds with single complexes (one soil group and one cover) and that watersheds were found for most of the listed complexes. An average CN for each watershed was obtained using rainfall-runoff data for storms producing the annual floods (USDA NRCS, 1993). The NRCS has never published the range of precipitation values used in the annual floods and therefore there are concerns as to which storm sizes are valid with the method (Hawkins et al., 1985).

Also, the scatter of the data on a P versus Q (where Q represents direct runoff) graph was attributed to the initial soil moisture. The watersheds used for the development of the method were generally less than 1 square mile in size, the number of watersheds for a complex varied, and the storms were of one day or less duration. The CNs of watersheds with the same complex were averaged and all CNs for a single cover were plotted on the P versus Q graph. "A curve for each cover was drawn with greater weight given to CN(s) based on data from more than one watershed, and each curve was extended as far as necessary to provide CN(s) for ungaged complexes" (USDA, NRCS, 1993). All this was done with less than 50 watersheds located throughout the county prior to 1954. The CN table has not significantly changed since its development in 1954, but supplementary tables for special regions have been developed. Nonetheless, the CN is a design tool in which its practical use in industry is beyond doubt. Therefore, a method was needed that could consistently determine CNs from gaged data, when available.

In order to determine a consistent method to derive a CN from actual watershed data, a method was developed by Hawkins (1993) which allows for the direct quantification of a gaged watershed's CN called the CNinfinity (CNinf). The term CNinfinity has never been published and is still in a research phase itself. However, the origin of the CNinf can be traced back to two publications, Hawkins (1993) and Hjelmfelt (1980).

The procedure to determine the CNinf is to first collect event precipitation and runoff by some non-subjective method. The event runoff volumes and the event precipitation depths are then independently ranked in descending order. This creates frequency matching between the precipitation and runoff data. Using the ranked data, the CNinf is then computed solving the quadratic form of the original NRCS runoff equation, which for any ranked P:Q pair ($0<Q<P$) results in a solution for CN through S, the potential maximum retention of precipitation during an event. Because this is a data derived value, it will almost never be constant (Hawkins, 1993). In the quadratic form of the runoff equation, the negative root is used to maintain the identity of $P = Q$ at $S = 0$. There is still a large degree of variation in the CN when determined in this manner.

Hawkins (1993) discovered that the data form a secondary relationship to precipitation when the ranked derived CNs, are plotted against the ranked precipitation. Hawkins found three common types of data behavior in the relationships: complacent behavior, the standard response, and the violent response. This variation requires that a systematic or uniform method is used to estimate the CNinfinity value.

Upon review of available data, it was decided that a CNinfinity could be extracted for a known USGS gage located within the Spring Creek Watershed. This value could than be compared against a CN determined using "standard engineering practice" to assess the need to adjust standard CN's within the Spring Creek Watershed.

The Stanlit3 program was used to compute the CNinfinity. The program uses a least squares fit of the quadratic form of the NRCS runoff equation. Solution of the least squares procedure is still variable at this stage of the method's development. A basic trend that becomes evident is that data derived CNs decrease with increasing precipitation. This was originally observed by Hjelmfelt (1991) and is not simply a result of ranking the data.

It is noted that from previous research (Fennessey, 2001), it was discovered that gaged data derived CNinfs should never be used in a design scenario or in the NRCS model. The reason is that CNinf values are typically higher than both table determined CNs and the actual CN used for design purposes to most closely match the historical data ("best" CN). The CNinf is thought to define a watershed's ultimate (no return period inferred) CN value regardless of land use. Therefore, although the CNinfinity is a value that can be used to represent a watershed's P to Q transformation, it in no way is directly applicable to be used in the current form of the NRCS runoff model. However, it can be adequately used to determine if a CN for an ungaged watershed is high or low.

In order to get a feel for a reasonable CN for the Spring Creek Watershed, a CNinfinity value was computed for the USGS gage at Houserville. Average daily flows provided by the USGS were converted to daily runoff volumes, while daily precipitation depths were taken from the Penn State University precipitation records from the Walker Building. Eighty events were extracted for the 10-year period from 1986 through 1996. Additionally, three of the 80 precipitation events used were checked using USGS breakpoint data for the gage and hourly precipitation data from the Walker Building, and it was found that the use of daily data adequately represented the transformation for CNinfinity extraction. The CNinfinity value computed was 66, while the traditionally computed CN value for the watershed using land use, soils maps, and the CN tables was 79. Figure E.16 shows the CN versus event precipitation depth for the data used plus the CNinfinity through the data.

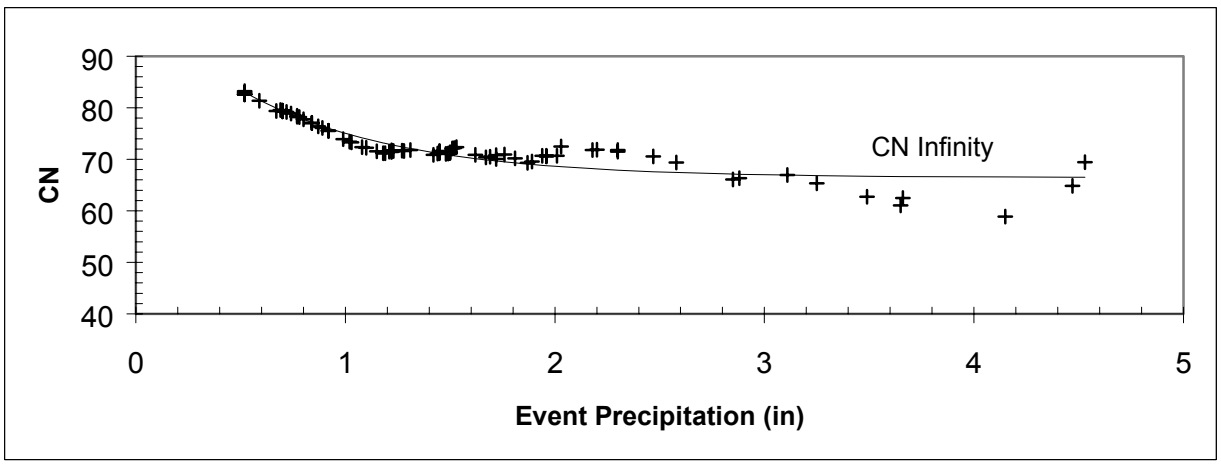


Figure E.16
Curve Number Infinity Determination from Houserville Gage Data (1986 to 1996)

The fact that the traditionally computed CN for the watershed is 13 CN values higher than the CNinfinity is significant because it is known that the CNinfinity is usually higher than the CN, which should be used in a model to best represent a watershed's direct runoff component. This provides reinforcement that the CN for the Spring Creek Watershed is high, most possibly due to the influence of carbonate geology, and may need some empirical method to reduce or adjust the CN for use in the TR-20 model. These results concur with the previous finding regarding the hydrologic response of the watershed.

Unit Hydrograph

The concept of the unit hydrograph was proposed by Sherman in 1931, but did not become founded in hydrologic procedures until it was merged in the late 1940's with Horton's infiltration excess theory. The NRCS fully developed this unit hydrographing concept for use with the soil cover complex model by developing a dimensionless unit hydrograph that is now typically used as a default in the TR-20 model and for most land development design. The NRCS's dimensionless unit hydrograph and how it is related to stormflow runoff volume and the time of concentration can be seen in Figure E.17. There are several underlying assumptions to the unit hydrograph concept. However, for the practical modeling application used in this study, there is only one important determination that needs to be made, and that is whether the NRCS's dimensionless unit hydrograph can be used to adequately represent a watershed. This can be checked using actual continuous precipitation and runoff data. If high quality data exists, a fairly accurate and somewhat consistent unit hydrograph can be extracted from actual data for a watershed. It was, therefore, one of the objectives of this study to verify that the NRCS dimensionless unit hydrograph could be validly applied to the Spring Creek Watershed for flood event modeling.

The USGS gage at Houserville was again used for the unit hydrograph determination. Breakpoint (random minute intervals) stage data provided by the USGS were converted to hourly runoff volumes in inches, while 1 hour precipitation depths were taken from the Penn State University precipitation records from the Walker Building. The runoff event of record, which occurred in January 19, 1996, was used. Although, this storm event could be considered a snowmelt runoff event for the area, it represents the type of storm, which causes the majority of the partial series floods in the Spring Creek Watershed. However, it could be reasonably

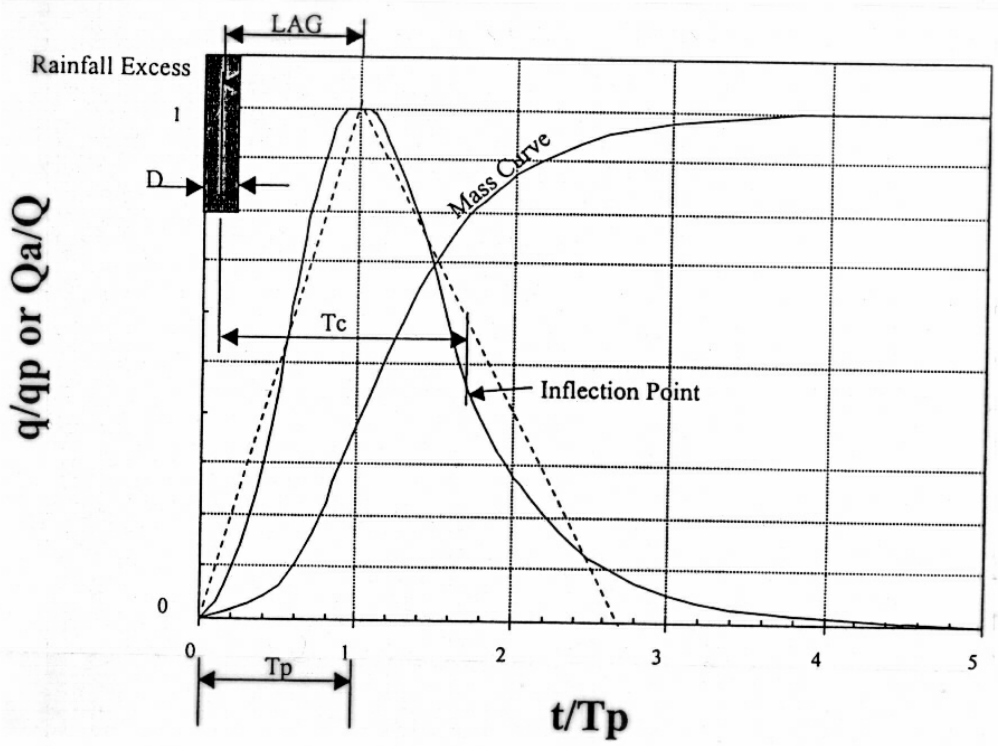


Figure E.17
NRCS Dimensionless Unit Hydrograph

argued that this unit hydrograph would not adequately represent the majority of minor water quality events.

The baseflow separation method was done using a method first proposed by Hewlett and Hibbert (1967). The method assumes that baseflow could be represented by a linear function with a constant slope. Two different slopes were used for the baseflow separation estimation. The first slope was selected as 0.0002 in/hr/hr as determined by Dripchak (1992), while the second was visually determined to be the best straight line slope based on judgement and experience. This selection resulted in a slope of less than 0.000 in/hr/hr. Generally, the base flow is considered to consist of less than 10% of the direct storm runoff, and therefore the method selected is considered negligible. Nonetheless both of the above methods were used. Although the variation in using the two different methods above was actually approximately 25%, this still did not significantly alter the shape of the unit hydrograph or the lag and excess precipitation hyetograph duration.

The end result of extracting the unit hydrograph was that when the dimensionless unit hydrograph for the Houserville gage was compared to the NRCS dimensionless unit hydrograph, the two compared as well as could be reasonably expected (refer to Figure E.18). It was therefore determined that the unit hydrograph should not be altered in the TR-20 model to account for the influences of carbonate geology for flood flow determinations. If it is later determined that the dimensionless unit hydrograph should be altered for common precipitation events for use in water quality estimates, the data and methods are available.

The USGS Houserville runoff data and the PSU precipitation were analyzed for the full year of 1996 using the GetPQ (version 1.5) model developed by the University of Arizona in conjunction with the USDA-Agricultural Research Service (ARS). The data was able to run through the program once modified for format. For 1996, over 70 independent storm events were identified, consisting of runoff hydrographs and rainfall excess hyetographs. These outputs supported the general shape of the dimensionless unit hydrograph for major events. However, there was some indication that the shape of the unit hydrograph may be dampened for more frequent runoff events.

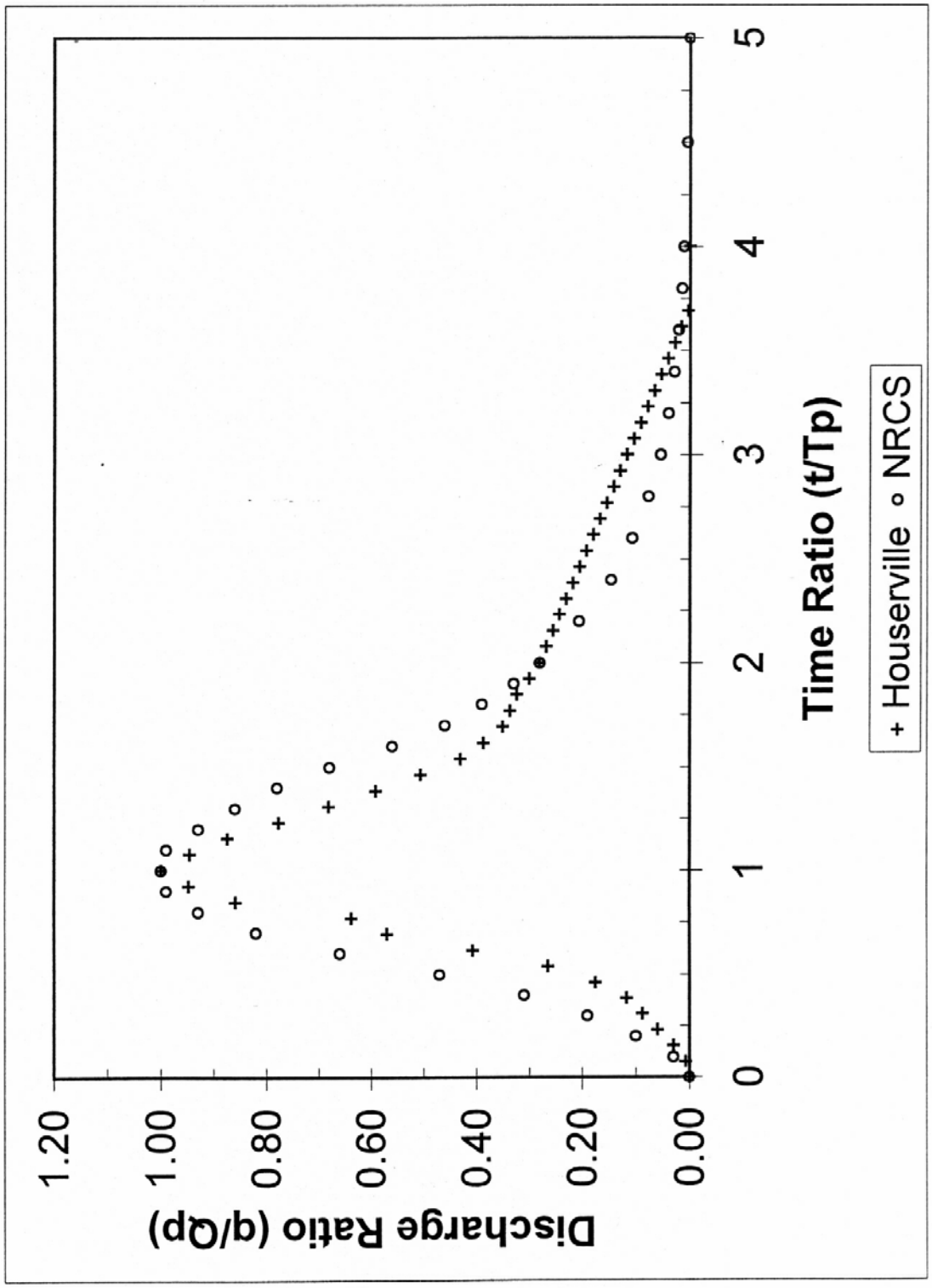


Figure E.18
Comparison of NRCS and Houserville Gage Dimensionless Unit Hydrograph

Determining a unit hydrograph for the frequent runoff events is beyond the present scope of work. If required, the unit graph could be modified. The following section provides information to the reader defining how the NRCS developed its dimensionless unit hydrograph. The NRCS developed their dimensionless unit hydrograph "from a large number of natural unit hydrographs from watersheds varying widely in size and geographical locations." The basic equation for the peak rate of the hydrograph was:

$$qp = \frac{K' AQ}{Tp}$$

Where: K' = constant
A = drainage area (square miles)
Q = direct runoff (in). This is the area under the hydrograph curve.
Tp = time to peak (hr)
qp = peak runoff rate (cfs)

From the observed data, the SCS derived an empirical relationship between the recessive limb of the hydrograph, Tr, and the rising limb of the hydrograph, Tp. The relationship was:

Tr = 1.67Tp and therefore K' = 484

where: K' = 645.33K

where: is for unit conversions from (Square miles-in)/hours to cfs
and K = 0.75 for a triangular hydrograph

The constant K' of 484 reflects a unit hydrograph that has 3/8 of its area under the rising limb. For mountainous watersheds the fraction could be expected to be greater and, therefore, the constant may be near 600. For flat swampy areas the constant may be in the order of 300 (USDA SCS, 1993; McCuen, 1982). These adjustments to the K' factor are not permitted or known by most regulatory agencies; therefore, the average condition is always used. This lack of adjustment in the K' value is a source of error in the prescribed stormwater regulations and can lead to an increase or decrease in the computed runoff peak in comparison to actual conditions. TR-55 (1986) does not allow the model user to change the constant K' from 484. Additionally, most users of TR-55 are not capable of adjusting or deriving a dimensionless unit graph based on a value other than 484.

The peak rate of runoff from a watershed is a function of watershed area, depth of runoff or direct runoff, Q, and a timing parameter. The timing parameter most often used for small watersheds is the time of concentration, Tc, which is often confused with the time to peak, Tp, and/or the watershed's lag time. The time of concentration is the time it takes for runoff (surface runoff) to travel from the hydraulically most remote point in the watershed to the watershed outlet or point of interest. In hydrograph analysis, the Tc is defined as the time from the end of excess rainfall to the point of inflection on the recessive limb of the hydrograph. For a description of different types of hydrographs used by the SCS- or CN-based models refer to Chow (1964).

The Tc can be shown to be related to the Tp by the expression:

Tp = 2/3Tc; (for small variations in the unit storm duration)

Therefore, Tc can be substituted for Tp resulting in:

$$qp = \left(\frac{3}{2}\right)\frac{484AQ}{Tc}$$

Where: A = drainage area (square miles)
Q = direct runoff (in). This is the area under the hydrograph curve.
Tc = time of concentration (hr)
qp = peak runoff rate (cfs)

The above equation shows the direct relationship between the peak runoff rate and time of concentration. Large variations in the computed peak runoff rate could be made due to variations in either Q or Tc. Errors in the delineation of the area of a watershed can generally be assumed small. However, for portions of the Spring Creek Basin this may not be the case.

Finally, from the previous studies mentioned above, it should be noted that the components of a runoff hydrograph located within the carbonate areas of the Spring Creek Drainage Basin are not typical of most areas. Figure E.19 shows the traditionally accepted components of a non-karst watershed, while Figure E.20 shows the general components for the majority of the Spring Creek Basin. As can be seen from comparing the two figures, the Spring Creek baseflows are higher than a traditional watershed, but more importantly, the subsurface runoff component makes up a greater percentage of the total storm runoff than the surface runoff component.

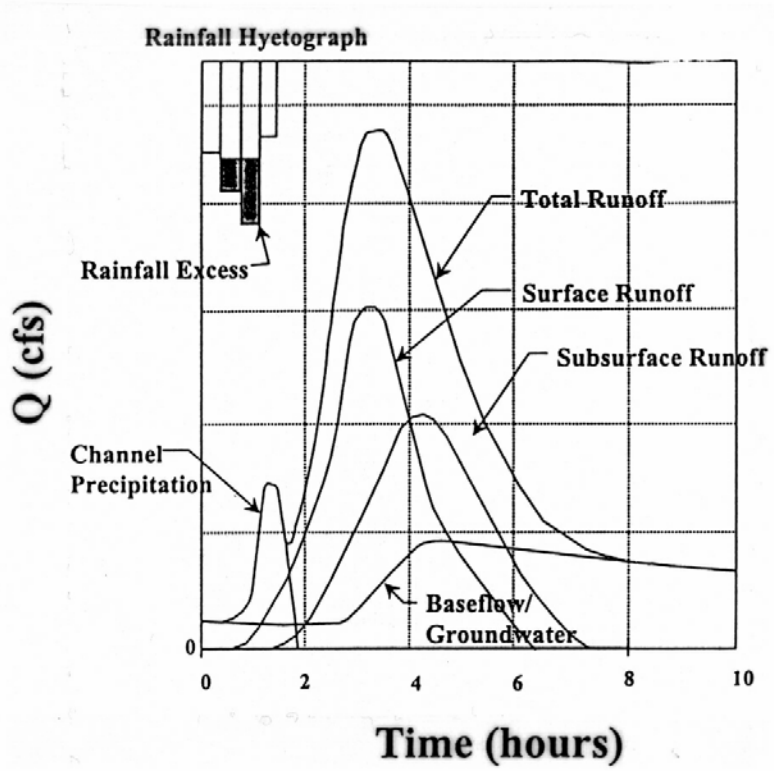


Figure E.19

Components of a Traditional Runoff Hydrograph

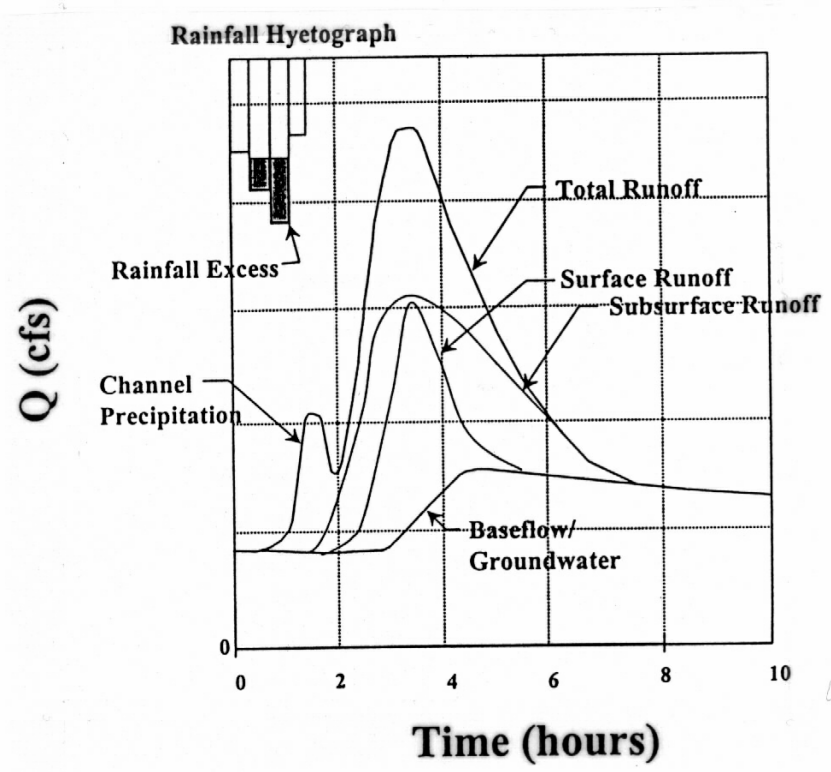


Figure E.20

Components of a Spring Creek Runoff Hydrograph

This difference would have a significant impact on the choice of the method to use for determining the watershed's time of concentration.

Time of Concentration

The method used to compute the time of concentrations (Tc) for the TR-20 model simulations in this study were based on the NRCS' original Lag equation. Although the Lag equation's reputation and use has become somewhat tarnished in the last 20 years, it is believed to be a more appropriate computational technique for the estimation of the pre-development scenario often associated with land development projects. There are four primary reasons the Lag equation was used in this study. First, the equation mimics the segmental equation for small watersheds up to approximately 50 acres (once larger than 50 acres, the NRCS's segmental method is know to under-predict a watershed's Tc, thereby overestimating pre-development runoff rates). Secondly, the Lag equation can not be "manipulated" as easily as the segmental method and generally, two different people using the equation will compute approximately the same Tc (which is not the case with the segmental method). The third reason the lag equation was used as the basis of determining the watershed Tc is because of the equation's simplicity. Finally, the carbonate geology dictates that infiltration excess is often not the major runoff process in the Spring Creek Watershed for natural areas, and therefore the idealization of the segmental method, which does not account for interflow mechanisms, is unjustified. Additional justification is developed in the following pages, which is a historical summary of the development and alterations of the NRCS's methods for computing Tc.

During the extraction of a unit hydrograph, a Lag equation based Tc of approximately 6 hours was found for the Houserville Watershed. This is within a very close range of what one computes using the Lag equation for the watershed (a lag based Tc of 5.46 hours was initially computed for the Houserville Watershed using a 5 subarea model). The segmental equation, because of its structure and assumptions, would produce a Tc of less than two hours for the same watershed.

For small watersheds, the method to compute the time parameter used to determine the peak rate of runoff in the NRCS soil cover complex method has gone through several changes. The original NRCS method was the Lag equation. The first published method, in 1968, to determine

peak runoff rates for small watersheds was with SCS-TP-149, which used the Lag equation to generate charts from which peak runoff rates could be estimated. Later the SCS published TR-55 (USDA SCS, 1975) and TR-55 (USDA SCS, 1986). Miller and Woodward (1995), in reference to the segmental method proposed in the 1986 version of TR-55, stated that the change to the segmental method made the time of concentration determination a more physically based approach. Although we do not concur with this statement, to best understand the changes that occurred regarding the determination of the hydrograph time parameter methods, a review of the three primary SCS small watershed documents (SCS-TP-149, 1968; TR-55, 1975; and TR-55, 1986) will be made

In an unpublished study, Fennessey (1997) showed that, for 24 mostly undeveloped (agricultural and natural) watersheds ranging in size from 20 acres to 1,950 acres, in every case the TR-55 (1986) segmental Tc method computed lower times of concentration for the watersheds than the Lag equation. The use of the segmental method to calculate Tcs resulted in larger peak runoff rates for every storm analyzed than by using the Lag equation. Several of the study watersheds had no channel networks and most likely were very similar to the original watersheds used to develop the Lag equation.

In another study using 37 actual gaged small watersheds from the ARS, Fennessey (2001) found that there was no difference between the two methods. This reinforced the fact that because there was no major statistical difference between using the Lag equation or the segmental method for the basis of the time of concentration, the Lag equation should almost always be used for the pre-development condition. The segmental method should still always be used for the post-development condition because it attempts to reflect the physical process of runoff from developed areas.

In addition, Kibler et al. (1982) noted that the SCS TR-55 Tabular Hydrograph Method has a disturbing tendency to produce higher peaks with increasing subdivision of the watershed. Considering the homogenous watershed assumption, it can be concluded that the only cause of the increase (not considering the general errors caused by TR-55's simplification of the CN method) is in the selection of the times of concentration for the watershed sub-basins.

SCS-TP-149 Lag Equation:

SCS-TP-149 did not directly use the time of concentration for peak flow rate computations. This is because it was developed for use primarily by SCS field personnel. The method instead presented ES-1026 graphs (for Type I rainfall distributions) and ES-1027 graphs (for Type II rainfall distributions), from which one determined the peak runoff rate according to the CN, average slope, and area of a watershed for a given total 24-hour precipitation. To use the method once the watershed CN had been determined, the user only needed to know the watershed area, average slope, and design precipitation event in order to predict the associated peak runoff. If the average slope varied from the chart value used, then interpolation could be used between two charts. Additionally, if the watershed length to width ratio varied widely from the typical data used to develop the method, then the effective area of the watershed could be adjusted. The November 1984 version of the Engineering Field Manual for Conservation Practices states, "the state conservation engineer may choose to develop an adjustment factor for a region or have the peak computed manually for an individual watershed if the shape factor is more than 50 percent greater than the following equation."

$$\frac{l}{w} = a^{0.2} \qquad \text{(Eq.16)}$$

Where: l = hydraulic length of watershed (ft)
w = average width of watershed (ft)
a = watershed area (acres)

The computation of peak runoff without a time of concentration variable was accomplished with the use of the Lag equation. The Lag equation was originally developed from the agricultural watershed data, which were used to develop the CN and is therefore often called the CN time of concentration.

The second [Victor Mockus altered the original equation following additional research (Mockus, 1960)] form of Lag equation is:

$$L = \frac{l^{0.8}(S+1)^{0.7}}{1900Y^{0.5}} \qquad \text{(Eq. 17)}$$

Where: L = lag, hours
Y = average watershed slope, %
l = watershed length, ft
S = maximum potential infiltration

The lag equation is no longer a method recommended by the NRCS to compute a watershed Tc if there is channel flow. Use of the equation has been reduced to estimating upland flows only. This seems odd, because if the data for development of the Lag equation were for watersheds up to 2,000 acres, then the author is not aware that a 2,000-acre watershed (in the northeast US) could not have a channel. Many land development ordinances will not allow the use of the Lag equation to determine Tc, even when the pre-development condition of the watershed is agriculture.

Using the combination of the Lag equation, the watershed length to width formula, and the CN relation to S, the Tc can be determined directly by the empirical equation:

$$Tc = 5/3L \qquad \text{(Eq. 18)}$$

and

$$l = 209a^{0.6}$$

and

$$S = \frac{1000}{CN} - 10 \qquad \text{(Eq. 19)}$$

Then (based on equation 17B):

$$Tc = \frac{5}{3}\left(\frac{(209a^{0.6})^{0.8}(\frac{1000}{CN} - 9)^{0.7}}{1900Y^{0.5}}\right) \qquad \text{(Eq. 20)}$$

Where: Tc = time of concentration, hours
L = lag, hours
a = area, acres
Y = average watershed slope, %
l = watershed length, ft
S = maximum potential infiltration
CN = curve number

"Slope as used in this method (the Lag equation) for computing peak discharge means primary average watershed slope in the direction of overland flow. Slope is readily available at most locations from existing soil survey data" (USDA SCS, 1993). Unfortunately, average slope calculation methods for watersheds often produce slightly variable results. SCS-TP-149 (1968) read, "On larger watersheds the gradient of the stream channel becomes an additional consideration in estimating time of concentration. An estimate of one average slope for all the land within watersheds of less than 2,000 acres is adequate for the slope parameter Y." This is in reference to agricultural watersheds.

TR-55 (1975) Average Velocity Method:

Prior to the development of a soil-cover complex version for urban watersheds, several general processes were known. First, it was clearly understood that the methods developed and reviewed for predicting the peak runoff rates in SCS-TP-149 or the ES-1027 charts were in no way appropriate for urban environments. Curve numbers had already been derived for non-agricultural conditions. However, it was also know that the modification of the hydraulic length of the watershed occurred from development by the changes to drainage systems. These changes generally consist of the installation of storm drain piping or channel improvements (increasing capacity and velocity) that decreased the watershed Tc. Additionally, development of a watershed implied the addition of impervious areas.

Therefore, when the TR-55 method was published in 1975, the changes in watershed hydrologic response due to development were considered. At the time, the Lag equation was still well received and was therefore the basis for the new model. Factors to adjust the Lag equation for both the hydraulic length modifications and the percent of impervious area were developed. These modifications were named the modified Lag equation. The original basis for this work is not known; however, in time these adjustments and those proposed by others were not recommended for use by the industry.

A large amount of research preceding the publication of TR-55 was directed at determining the flow velocities of water over varying ground cover conditions. These data were collected through the use of research plots. The data were then used to develop a method called the Average Velocity Method to determine the Tt within a watershed. In a personal communication

with Dr. Gert Aron, it was discovered that the results were highly inconclusive, except on paved surfaces. This method allowed the computation of the overland flow component as:

$$Tt = \frac{l}{3600v} \qquad \text{(Eq 21)}$$

Where: Tt = time of travel (hrs)
l = overland flow length (ft)
v = average velocity (ft/sec)

The time of concentration was then computed by summing the various Tts for the watershed and adding the travel time computed for channel flow. The use of Manning's equation was recommended for channel flow.

It could not be documented, but it is believed that the developers of the velocity method never checked the computed peak runoff rates, using the velocity method for computing the watershed Tc against the historical peak runoff rate data from the original watersheds used to develop the CN or the Lag equation. Additionally, the 1975 version of the TR-55 manual does not cite specifically from what research the average velocities shown on the chart were developed.

TR-55 (1986) Segmental Method:

In 1986, when the revised (2nd Edition) TR-55 manual was published, the Lag equation was completely removed from the publication. Additionally, the average velocity method was modified for simplicity. The new method became known as the Segmental Approach. It was called the segmental approach because the watershed Tc consisted of three components; sheet flow, shallow concentrated flow and channel flow. The method was similar to the average velocity method with two major changes: 1) modifying the average velocity chart to reflect only paved or unpaved surfaces; and 2) adding a sheet flow component. Research had shown that surface runoff generally started as a sheet of water flowing over the land, which after a certain period of time formed small channels of water or shallow concentrated flows; and if the flow length was long enough, the shallow concentrated flows would converge into channels which could be modeled using Manning's equation.

As previously mentioned, this method was not believed to be validated for the agricultural watersheds that were used to derive the CN method. The segmental method was developed using controlled research plots because only limited data existed for urban watersheds. This change has staggering implications if one considers that the original CN method was meant to predict the direct runoff of a watershed, because direct runoff consists of surface runoff and subsurface runoff. One of the largest problems with the calculation of the Tc by this method is it assumes all runoff is surface runoff. This can result in increased computed peak runoff rates, especially for agricultural or slightly urbanized areas using the original CN values.

The sheet flow component was developed by Overton and Meadows (1976) and was defined as:

$$Tt = \frac{0.007(nL)^{0.8}}{P_2^{0.5} s^{0.4}} \qquad \text{(Eq. 22)}$$

Where: Tt = travel time (hr)
n = Manning's roughness coefficient for sheet flow.
L = flow length (ft)
P_2 = 2-year, 24-hour rainfall (in)
s = slope of the hydraulic grade line (ft/ft)

The TR-55 manual for sheet flow allowed a maximum length of 300 feet. This was based on work published by Engman (1986). In regard to the maximum slope length for sheet flow Engman states, "These data should be valid for so-called sheet flow or shallow-depth overland flow that match the conditions of the experimental plots. The user must limit the flow plane lengths. It appears that excessive depths would not be encountered if the slope lengths are on the order of 150 to 300 feet."

Shortly after publication, regulatory agencies (the original source was not located) stated that in the northeast, for unpaved areas, the maximum sheet flow length that would occur is 150 feet and more likely in the range of 50 to 100 feet. Regulators quickly modified TR-55 for their own use by not allowing the computation of sheet flow lengths over 100 feet. Although this is physically reasonable, the implications on the total Tc or conversely the peak runoff rate were never explored in relation to the original model. In small watersheds, the Tc is comprised largely by the sheet flow component. Additionally, since most engineers do not have adequate data, a sheet flow length of 100 feet is almost always used for all watersheds.

The use of 0.80 for the Manning's roughness coefficient for woods with dense underbrush also later came under fire. TR-20 (1982) supplementary literature states, "The maximum roughness coefficient for sheet flow to be used for woods is 0.40. The data does not support the higher coefficient." Engman's (1986) data did not discuss wooded conditions. Where the original value of 0.80 came from is unknown.

As stated previously, the average velocity for overland flow was simplified for only paved and unpaved areas. The selection of 'n' and hydraulic radius values by the SCS for use in determining overland flow velocities with the Manning's equation are very limited. These values were selected assuming practitioners did not have accurate watershed data or did not fully understand the model. Nonetheless, comparing the average velocity chart from the 1975 TR-55 manual with the average velocity chart from the 1986 version, we observe that for an n = 0.05 (fallow land) the velocities one gets from the two charts are not similar (for the same land cover), and the line on the 1986 TR-55 average velocity chart for "unpaved" actually would fall between the lines for grassed waterway and paved area (sheet flow) and shallow gutter flow on the 1975 average velocities chart. The consequences again are that the Tc is less for a watershed modeled with the 1986 version of TR-55 than the 1975 version, which was confirmed by Fennessey (1997). This again would generally lead to an increase in computed peak runoff rates.

Therefore, from this study and previously reviewed work, the Lag equation is proposed as the method to compute all pre-development time of concentrations. The segmental method should still be used for a post-development Tc computation. However, because of its poor use and manipulation by users conducting stormwater management studies, later recommendations will be provided for its correct use.

Model Calibration

Following the analysis of the Watershed hydrologic data and the matched pair analysis, it was believed that the TR-20 model could be modified to adequately reflect runoff from the Basin and the Shiloh Road Watershed for engineering design purposes.

Shiloh Road Sub-Watershed:

A three-subarea Shiloh Road watershed model was developed for the initial tests using the TR-20 model. The separation between Subareas 1 and 2 was due to the proposed location of Interstate-99. In order to test the effect of the hypothetical boundary between Subareas 1 and 2, a two-subarea model was also run which did away with this hypothetical boundary. Comparing the two models, it was determined that the three-subarea model produced similar peaks rates and could be used. The differences ranged from 9.6% for 2-year runoff event and 1.5% for the 100-year runoff event. Subarea 3 consisted of mostly developed areas within the watershed that are directly connected impervious areas. However, this developed area is conveyed to a known stormwater management facility in which the actual stage-discharge data were used in the model.

Similar to the matched pair analysis, the Shiloh Road Watershed was modeled using PSU-IV with and without carbonate adjustments, and TR-20 with standard methods used for determining the input parameters. Standard methods imply that the curve numbers used in the model were determined by actual land use and the NRCS hydrologic soil group (HSG) classification, and the timing method used was the NRCS's segmental method. A plot comparing the three modeling results can be seen in Figure E.21. Comparing this plot to the two plots from Pair 1 of the matched pair analysis (Figures E-22 and E-23), the TR-20 model estimates appeared clearly too high, while the PSU-IV estimate with carbonate adjustments appeared to be too low for the higher return period events. The TR-20 estimates for Shiloh Road are higher than the two Pair 1 watershed TR-20 estimates because of HSG group differences and because the Lag equation was used for the timing method in the initial matched pair analysis (refer to the report section on the matched pair analysis). The comparison can also be seen in Table E.2.

In order to reinforce the belief that the TR-20 estimates were initially too high for the Shiloh Road Watershed, the estimates were plotted against the historical data for both the Axemann and Houserville Watersheds (refer to the previous Figure E.13). Although a decay of peaks is expected going from a first order (Shiloh Road Watershed) to a tertiary channel system (Spring Creek), the Shiloh Road estimates are still unreasonably high considering that the size of the three watersheds is 1,000 acres, 37,440 acres, and 55,808 acres for the Shiloh, Houserville,

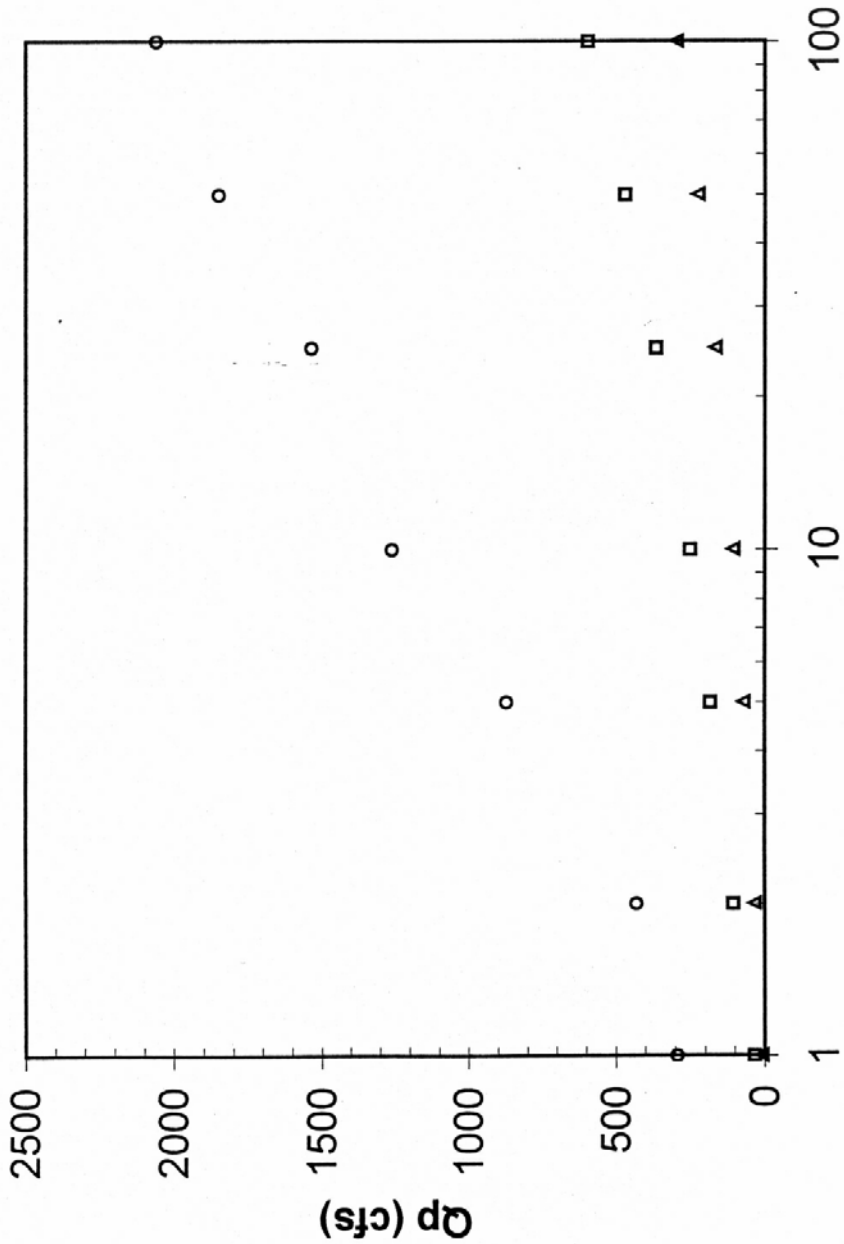


Figure E.21
Comparison of Model Estimates for Shiloh Road Watershed

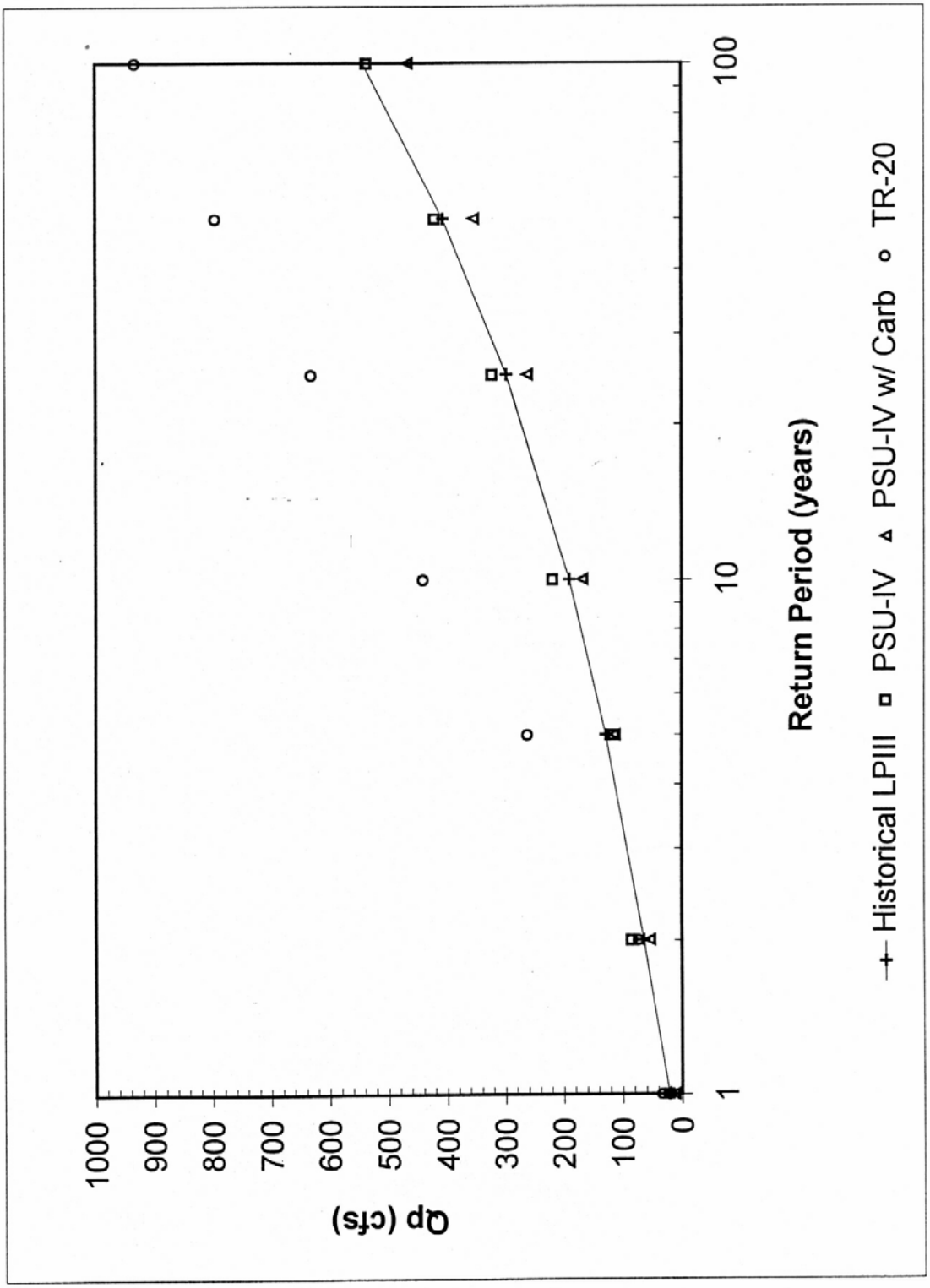


Figure E.22
Comparison of Model Estimates to Historical Runoff for Bull Run Near Loganton

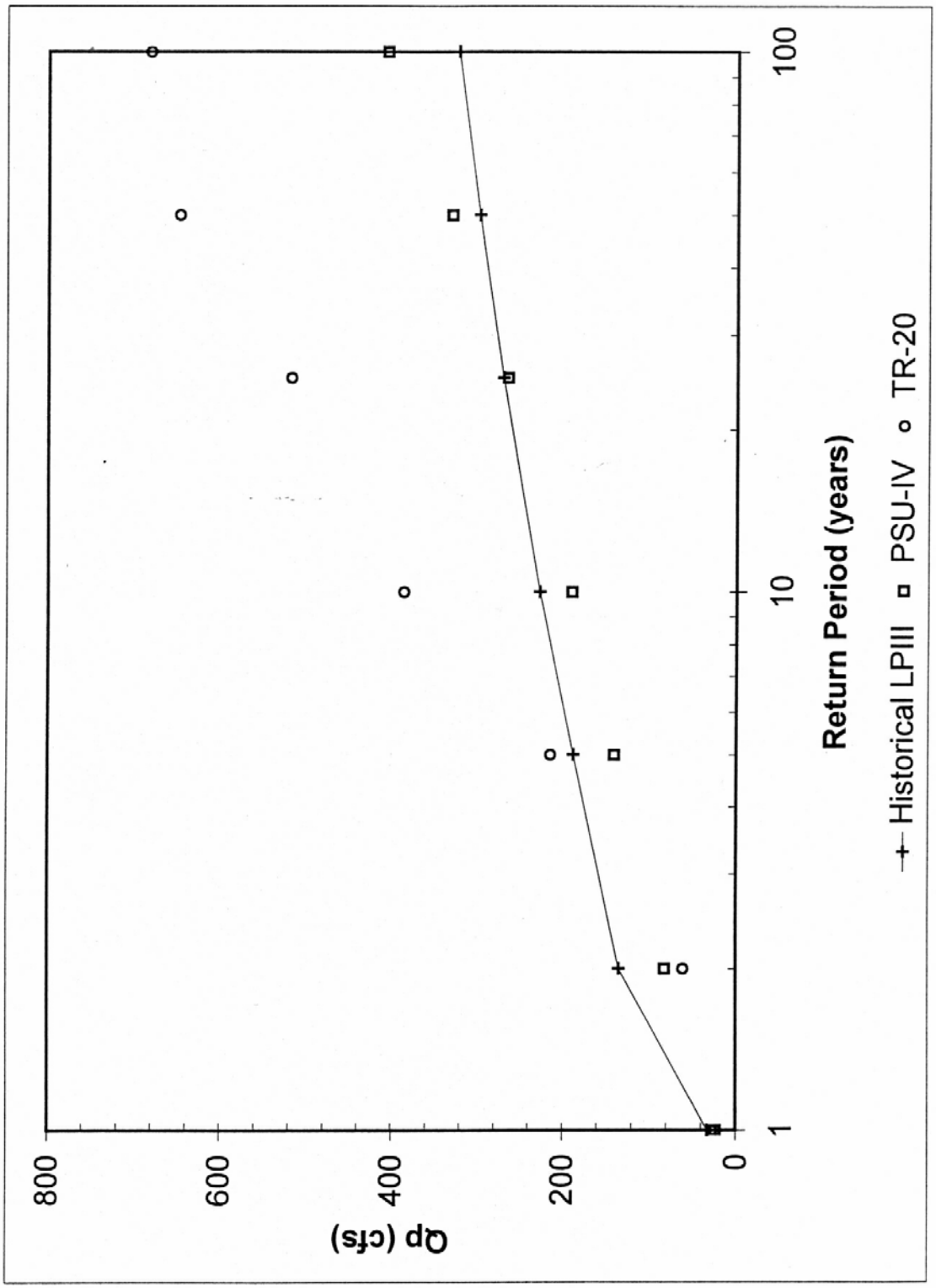


Figure E.23
Comparison of Model Estimates to Historical Runoff for Schell Run at Tyrone

Table E.2
Comparison of Pair 1 to Shiloh Road Watershed

PAIR ID	Gage #	Sq. Miles	Data	Peak Runoff Rates (cfs)						
				1	2	5	10	25	50	100
p1	1557100	1.68	Historical	50	120	187	227	270	298	323
p1	1557100	1.68	PSU-IV	23	82	140	189	264	330	405
p1	1557100	1.68	TR-20	27	60	214	385	517	647	682
k1	1548020	1.99	Historical	27	65	128	190	298	406	542
k1	1548020	1.99	PSU-IV	22	84	115	219	323	420	536
k1	1548020	1.99	PSU-IV w/k	13	54	113	168	262	354	466
k1	1548020	1.99	TR-20	33	73	262	441	631	795	933
SHILOH	na	1.99	Historical	NA	NA	NA	NA	NA	NA	NA
SHILOH	na	1.99	PSU-IV	33	104	181	250	363	468	595
SHILOH	na	1.99	PSU-IV w/k	10	35	72	107	167	225	297
SHILOH	na	1.99	TR-20	293	429	870	1259	1535	1848	2059

and Axemann Watersheds respectively. Refer to the report section on seasonal variations in the Spring Creek Drainage Basin for additional comments on this subject.

Field investigations conducted at the Shiloh Road downstream channel section also reinforced the opinion that the initial TR-20 estimates were too high based on the downstream channel size, shape, and stability. The most substantial reasoning that the initial TR-20 model estimates were too high was based on personal experience. The Authors have intensely analyzed and studied data from hundreds of gaged watersheds throughout the United States, and from practical experience realized the initial TR-20 estimates were of an unreasonable order of magnitude for a watershed of its size even without considering its carbonate geology.

Although no historical gage data exists for the Shiloh Road Watershed, several attempts were made at actually gauging runoff on the watershed using an existing weir located near the outlet of the watershed. During September 17 and 18, 1999, field observations were made due to the possible precipitation expected from Hurricane Floyd. Although the expected precipitation depths were not received, approximately 0.88 and 1.26 inches of rainfall fell over one and two days respectively. Using the initial TR-20 model structure, estimates of between 9 and 60 cfs of runoff could be expected from the Shiloh Road Watershed for similar synthetic design events. However, no runoff actually reached the outlet during the event. A similar condition occurred on September 29, 1999 when a high intensity storm event of 1.55 inches occurred over the State College area in 24 hours. Again, like the previous event, no runoff reached the outlet of the watershed even though the upstream developed area's pond was discharging approximately 0.5 cfs for most of the following day. Additionally, although no surface runoff occurred from the Shiloh Road Watershed, direct runoff hydrographs were observed at the Houserville gage for both events.

It is believed that these initial Shiloh Road Watershed estimates are high because the standard computational methods were used to determine the input parameters. In other words, the HSG and land use, were used to determine the CNs, while the segmental method was used for the time of concentration determination. Using the analysis of the Houserville Watershed, it is clear that the NRCS dimensionless unit hydrograph was acceptable for major storm event modeling, and therefore should be left alone (refer to the report section of the Houserville unit hydrograph). We were also able to conclude that the CN was too high for the region and should be adjusted.

This conclusion has also been observed in other karst areas. Bonacci (1987) stated that in karst terrains 'the possibility of surface flow is either unlikely or completely eliminated' and that 'the covering soil layer in karst, from a hydrologic point of view, has a limited influence on the formation of the processes of surface, subsurface, and groundwater flows.' Therefore, it was decided to adjust the concept of the CN use without actually adjusting the NRCS's CN table by simply adjusting the HSG that relates to a soil's infiltration capacity. This is a more viable option than using Antecedent Runoff Condition I (ARC I). The concept of the ARC variations have never been supported by any known data.

Additionally, in making adjustments to the NRCS's methods, it was desired to not alter the modeling requirements too radically from what engineers and designers are currently familiar. A common practice already used in several Centre County municipal ordinances is to model all areas as being either meadow or woods in good condition for pre-development conditions regardless of the actual land use. From previous research, it was also noted that it would be better if the NRCS's Lag equation was used for the time of concentration (Tc), instead of the segmental methodology because it results in more uniform and consistent numbers between different users and results in Tcs that are no different statistically than those from the segmental method for small watersheds. Both of these adjustments were independently made and combined together for three variations to the way TR-20 was modeled for Shiloh Road; however, the results still appeared too high using the PSU-IV estimates as a baseline.

It was therefore decided that since the overall effect of the carbonate geology on peak flows was to attenuate them, this could be mimicked by reducing all of the HSGs to Group B soils. Again, as Bonacci (1987) noted this is not an unrealistic assumption. The logic behind this is that while most soils in the Shiloh Road Watershed, and also the Spring Creek Watershed, are classified as HSG C soils, the infiltration capacity of the soils, because of their carbonate influence, probably more nearly respond like HSG B or HSG A type soils. This adjustment was

also incorporated into the two previous adjustments for a total of seven possible variations as:

mod 1 = CN all meadow or woods with actual HSG
mod 2 = CN all meadow or woods with HSG B regardless
mod 2a = CN all meadow or woods with HSG B except for existing impervious areas
mod 3 = CN all meadow or woods with actual HSG and Lag based Tc
mod 4 = CN all meadow or woods with HSG B regardless and Lag based Tc
mod 5 = lag based Tc only modification
mod 6 = mod 2 with planned development changed to impervious

In a personal communiqué with a representative of the NRCS, Scott Brown of Sweetland Engineering was informed that the NRCS had changed the HSG classification of Hagerstown Silt Loam soils from a HSG C to HSG B approximately five years ago. Although the USDA's Soils-5 database does not confirm this change, it would be a significant modification by the NRCS, if true; because approximately 26% of the Spring Creek Basin is made up of Hagerstown soils. Nonetheless, this change by the NRCS would further support our findings.

Prior to conducting the runs, a conceptual model output scenario was idealized. It was assumed that for small return periods, there should be almost no runoff from a watershed such as Shiloh Road. Secondly, it was assumed that for major (50-year and greater) events the runoff rates should be slightly higher than the PSU-IV estimates to account for the karst's ability to transmit high volumes of water quickly through conduit or diffuse flow.

The model results for each scenario are shown in Table E.3. From the results it appeared that the two apparent best model scenarios were mod 2 and mod 2a when all soils were considered HSG B with all land uses being assumed to be meadow or woodland. From the differences between these two runs, it is evident that allowing the impervious areas to be modeled as such, once correcting for the pervious areas, does not change the results significantly. From the table, it appears that scenario mod 4, which uses the Lag equation, is initially too low because the Lag equation is a function of the CN, which was already reduced in modifying the HSG. However, it is known that this difference will diminish once the watershed is more realistically subdivided into smaller components.

Once a degree of confidence was attained with the initial three-subarea watershed model, the watershed was then further subdivided into 31 subareas (refer to Figure E.24). The sub-

Table E.3
Comparison of Shiloh Road Watershed Model Variations

PAIR ID	Gage #	Sq. Miles	Data	Peak Runoff Rates (cfs)						
				1	2	5	10	25	50	100
SHILOH	na	1.99	Historical	NA	NA	NA	NA	NA	NA	NA
SHILOH	na	1.99	PSU-IV	33	104	181	250	363	468	595
SHILOH	na	1.99	PSU-IV w/k	10	35	72	107	167	225	297
SHILOH	na	1.99	TR-20	293	429	870	1259	1535	1848	2059
SHILOH	na	1.99	TR-20 mod 1	92	196	309	479	772	960	1225
SHILOH	na	1.99	TR-20 mod 2	14	34	72	146	307	431	623
SHILOH	na	1.99	TR-20 mod 2	17	43	90	172	352	484	687
SHILOH	na	1.99	TR-20 mod 3	66	128	193	290	462	574	732
SHILOH	na	1.99	TR-20 mod 4	13	29	51	90	169	226	316
SHILOH	na	1.99	TR-20 mod 5	217	358	487	656	926	1093	1324
SHILOH	na	1.99	TR-20 mod 6	79	165	265	413	679	852	1100

mod 1 = CN all meadow or woods with actual HSG
mod 2 = CN all meadow or woods with HSG B regardless
mod 2a = CN all meadow or woods with HSG B except for existing impervious areas
mod 3 = CN all meadow or woods with actual HSG and lag based Tc
mod 4 = CN all meadow or woods with HSG B regardless and lag based Tc
mod 5 = lag based Tc only modification
mod 6 = mod 2 with planned development changed to impervious

Figure E.24

11x17

watersheds were based on actual topographic divides and the existing drainage network. The number of sub-watersheds however, was influenced by the desire to be able to keep the same subarea structure for the future model simulations when development scenarios and BMPs would be tested. Scenario mod 4 was run with the 31-subarea model structure. As predicted, the estimates moved into line with the scenario mod 2 and mod 2a estimates from the three-subarea model. The 31-subarea model, using scenario #4 was therefore decided upon as the baseline estimate model. Two significant underlying assumptions in the model are the use of HSG B for all soils, and use of the Lag Equation for time-of-concentration estimates. All future model simulations will also be structured in the same manner. Refer to Table E.4 and Figure E.25 for the quasi-calibrated estimates that will from hereon be referred to as the existing baseline runoff rate estimate.

Table E.4

Final Model Adjustments Compared for Shiloh Road

Model	Peak Runoff Rates (cfs)						
	1	2	5	10	25	50	100
PSSU-IV	33	104	181	250	363	468	595
PSU-IV w/k	10	35	72	107	167	225	297
TR-20	293	429	870	1259	1535	1848	2059
TR-20 w/Adjustments	26	58	107	190	347	495	677

Houserville Sub-Watershed:

Analysis of the Shiloh Road sub-watershed suggests that hydrologic processes within the Spring Creek watershed can be simulated by making two (2) adjustments to standard modeling procedures. These adjustments include using HSG B for all soils, and the Lag Equation for time-of-concentration calculations. To determine how these proposed modifications alter runoff estimates for larger watersheds, a five-subarea model was developed for the Houserville watershed (refer to Figure E.26). Similar to the Shiloh Road Watershed, an initial model was developed using traditionally computed parameters, which radically over-estimated the peak runoff rates at the Houserville gage. Because only 15 years of data are available, the data were only compared for return periods less than 25 years. Although there is typically a large degree of variation in determining probability distributions for higher return period events, the historical one- and two-year return periods can be considered very reliable estimates. The traditionally computed peak runoff rates using TP-40 precipitation depths resulted in a 1 year return period

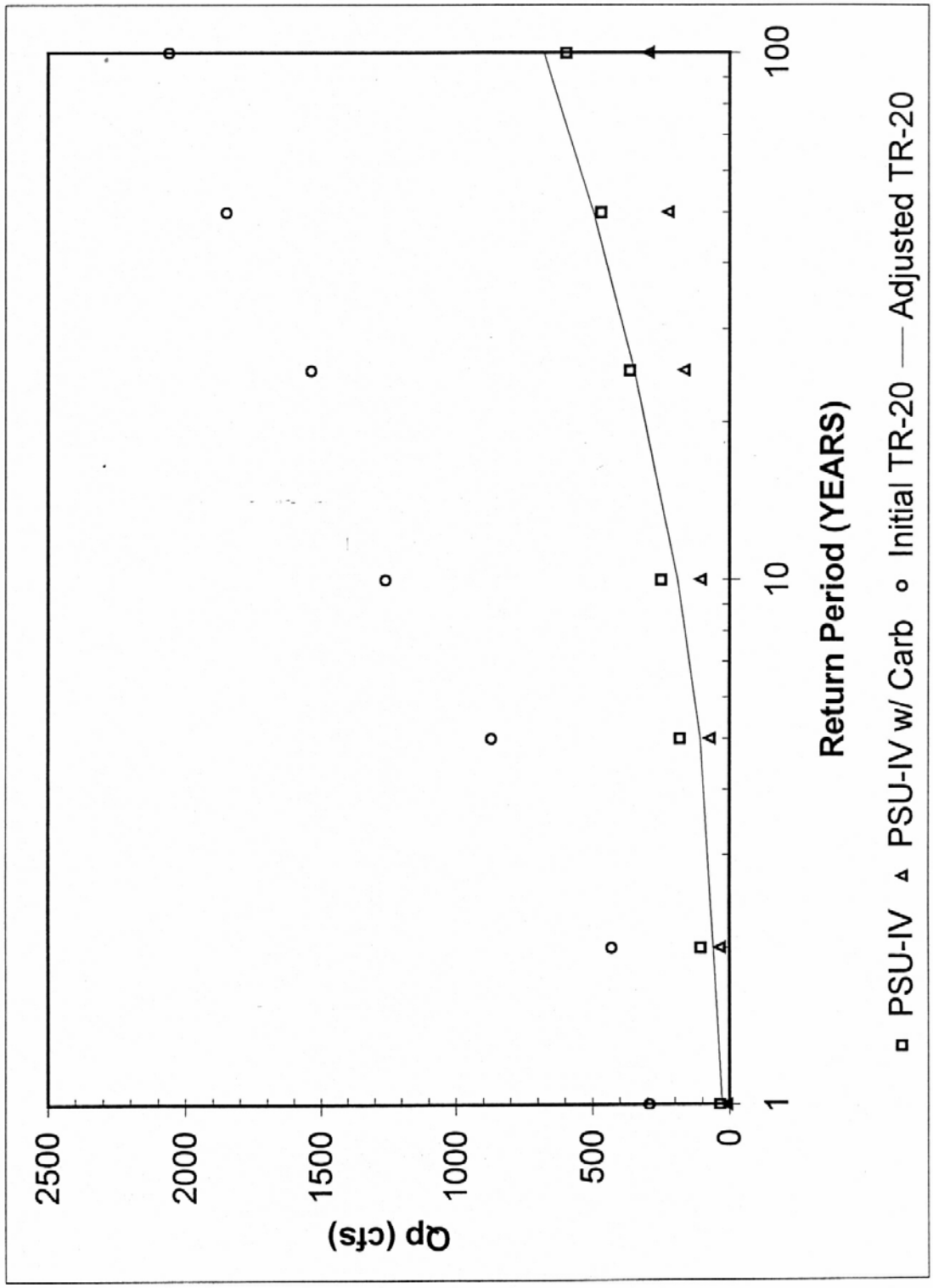


Figure E.25
Comparison of Model Estimated for Shiloh Road Watershed with Final Selected Adjustment

Figure E.26

11x17

peak rate model estimation for the Houserville gage that was 50% larger than the storm of record for the 15 years since the gage has been installed. This was while already using the Lag equation to compute the time of concentration. Had the segmental method been used for the time of concentration determination, the discrepancies would have been higher.

Although some people may argue that the watershed model could be calibrated to the gage data by adjusting other parameters, such as the hydraulic routings model parameters (modified Att-kin parameters x and m), this is not a realistic option because of the limited overbank storage within the basin. Therefore, the over-estimations were directly attributable to the selection of curve numbers for the watershed. Additionally, the unit hydrograph and time of concentration were known to be within an acceptable range in the model without change for extreme events, because of the previous historical analysis of the gage data at Houserville.

The curve numbers used in the model for the subarea watersheds were then re-computed using the modifications developed in the Shiloh Road watershed analysis. The model was then rerun with these modifications. These modifications resulted in more realistic estimates for the specific return periods. This agreement with the results of the Shiloh Road watershed analysis further support the model modifications in regard to the CN and time of concentration computational method proposed for use in the basin. A summary of these results is found in Tables E.5 and E.6.

Table E.5

Comparison of TR-20 Traditionally Computed Parameters Versus Parameters as Computed by the Proposed Standards within the Houserville Gage Watershed

Sub-watershed	Area (Acres)	Traditional Parameters		Modified Parameters	
		CN	Tc (Hours)	CN	Tc (Hours)
Roaring Run	4.72	73	2.02	57	3.12
Galbraith Gap Run	5.13	70	1.78	55	2.62
Slab Cabin Run	16.84	81	3.96	66	6.03
Cedar Run	17.47	80	4.30	60	7.63
Spring Creek	14.34	79	5.46	63	8.52
Roaring Run	21.174	83	6.33	65	10.33
Total	58.5				

Table E.6

Comparison of TR-20 Output Using Traditionally Computed Parameters Versus Using Parameters as Computed by the Proposed Standards within the Houserville Gage Watershed

Return Period (Years)	Peak Runoff Rates (cfs)		
	Historical Data	TR-20 Model with Traditional Parameters	TR-20 Model with Modified Parameters
1	600	3765	433
2	690	4,835	828
5	825	6,531	1,360
10	2370*	10,400	2,385
25		13,499	4,716
50	N/A	14,388	5,707
100	N/A	16,148	6,734

Summary

The historic data analysis and watershed simulations presented above provide substantial evidence of karst influences on hydrologic response within the Spring Creek Watershed. The following conclusions are drawn from this analysis relative to implementation of procedures which, include the influence of the karst characteristics of the region on hydrologic response:

1. The NRCS's TR-20 model is recommended for hydrologic simulations within the Spring Creek Watershed because of its wide acceptance by users, the ready availability of data and documentation, and because of its ability to be modified to account for the effects of karst influences.
2. To mimic karst influence within the watershed, all soils over carbonate substrata should be simulated as hydrologic soil group (HSG) B.
3. The general shape of the NRCS dimensionless unit hydrograph in TR-20 adequately simulates runoff within the watershed for major events. Adjustment of the dimensionless hydrograph shape is not recommended.
4. It is recommended that the Lag equation be used to compute all pre-development time of concentrations.

DESIGN STANDARDS

The second objective of the technical analysis is to establish design standards, which are consistent with natural hydrologic processes within the watershed to manage stormwater runoff and its impacts on flooding, groundwater recharge, channel stability, and water quality. The following sections consider these design standards.

Minimum Site Discharge

Because computational methodologies can sometimes result in pond release rates that may require either the use of extremely small principal spillway orifices or oversized ponds, minimum site discharge rates have been established. One reason is because extremely small principal spillway orifices often pose maintenance or clogging problems. Current requirements

sometimes require excessively large pond volumes to meet standards, which dictate the release of negligible flows (less then 0.1 cfs).

Current design criteria used for stormwater management ponds often make pond volumes ineffective while still causing downstream nuisance flooding. Therefore, the following minimum site discharges have been established based on drainage area:

1-year return period	$Qp_{min} = 0.018(DA) + 0.2$
2-year return period	$Qp_{min} = 0.03(DA) + 0.4$
10-year return period	$Qp_{min} = 0.09(DA) + 1.0$

Where : DA = the drainage area in acres
Qp_{min} = minimum peak discharge rate in cfs

There shall be no minimum discharges for return periods greater than 10 years.

The minimum peak runoff rate standards established above includes the total of all stormwater management facility discharges plus discharges from undetained areas. Undetained fringe areas (where the designer has made a realistic effort to limit all new impervious areas) will have the peak runoff rates computed using the pre-development time of concentration for the drainage areas that they are tributary to.

The drainage areas used in the equations above must be equal to the pre-development tributary areas in all cases and cannot include diverted drainage areas. For example, if in the pre-development analysis, one (1) acre drains to an inlet (which is considered the downstream point of interest), and after development the drainage area is increased to 1.5 acres, then the minimum discharge to the inlet must be calculated using the one (1) acre drainage area.

These minimum discharge rate equations were developed after conducting 7,920 TR-55 simulations using a mimicking program that greatly enhanced the run time. The 7,920 runs consisted of model parameters that represented various site shapes, area, slopes, land uses for the 1-, 2-, 10-, and 100-year return periods. The site areas were altered from 0.5 acres to 100 acres in size, the land use modifications resulted in varying CNs from 55 to 70, the average slope varied from 1 to 10 percent, and the hydraulic length (a measure of shape/size) ranged from 104 feet to 2,952 feet. The lag equation was used as the basis to determine time of concentrations, which ranged from 2.2 to 148.7 minutes for the sites. Graphs of the 7,920 runs with the minimum discharge lines can be seen in Figures E.27A through E-27C. These graphs

and a large array of existing site data from the actual sites within the Spring Creek basin were used to determine the finial minimum discharge functions presented above.

As a final check, the 10 actual sites that were used for the economic analysis presented in Chapter 6 were also tested using the minimum discharge equations. As seen in Table E.7, only three of the sites resulted in minimum discharge modifications and only one site had the minimum discharge applicable to all three return periods. Additionally, the site with the lowest 1-year peak rate discharge, 0.1 cfs, had an allowable minimum discharge of 0.3 cfs. This increase, although more than doubling the discharge, is still negligible and would help alleviate maintenance problems associated with designing ponds for extremely small discharges.

Recharge Volume

Development activities (particularly the addition of impervious areas) within watersheds change the hydrologic processes and create an increase in the volume of runoff. This results in a diminished groundwater recharge potential. This reduced recharge can significantly reduce the baseflow within perennial streams, and in some cases can alter the flow characteristics within stream reaches to resemble that of an intermittent stream during dry periods. Requiring a minimum recharge volume from each developed site helps ensure that baseflows and groundwater reservoirs can be maintained within the drainage basin following development activities.

Because of the carbonate geology within the Spring Creek Watershed, baseflow percentages are higher than similar non-carbonate watershed. Therefore, it was decided that an analysis should be conducted to evaluate recharge requirements from a quantifiable hydrologic perspective. The Axemann gage, located along Spring Creek, was selected for this analysis because of its extensive length of available data. Data from Taylor's 1997 study of the basin water budget were used for determining the baseflow percentages of the total flow and the annual precipitation depths. Peak discharge data was taken from the USGS database, and census data was collected from U.S. and Centre County statistical references.

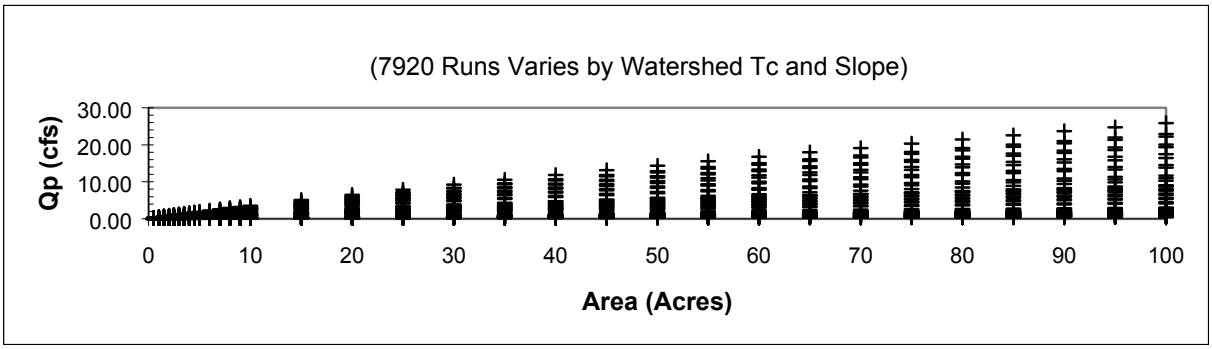


Figure E.27A
Comparison of Minimum Allowable Discharge Versus TR-55
Simulation Runs for 1-Year Return Period

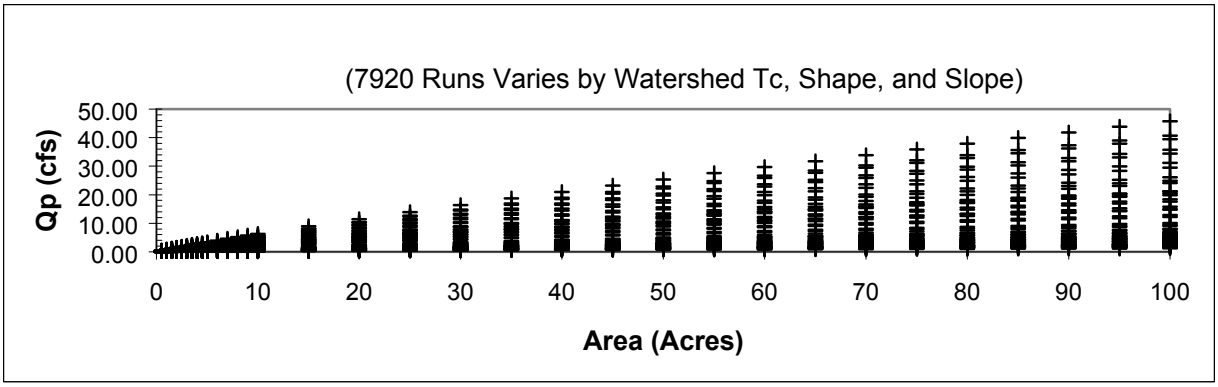


Figure E.27B
Comparison of Minimum Allowable Discharge Versus TR-55
Simulation Runs for 2-Year Return Period

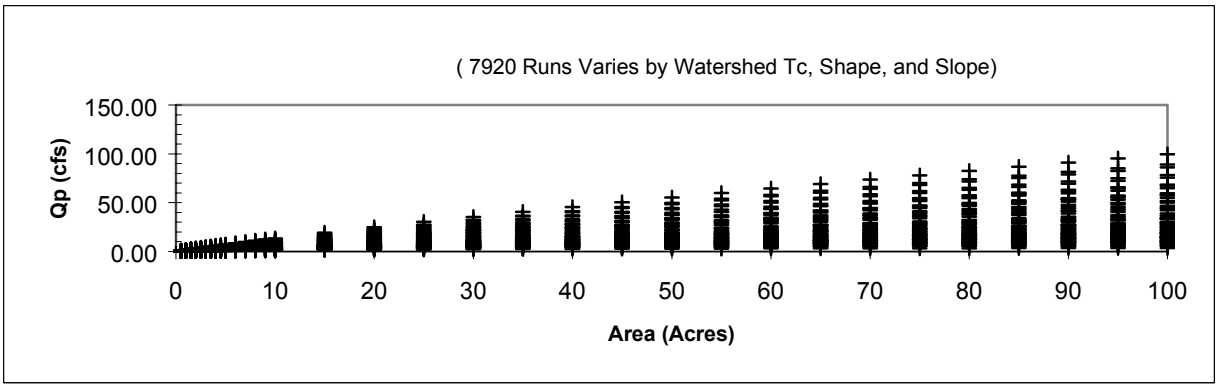


Figure E.27C
Comparison of Minimum Allowable Discharge Versus TR-55
Simulation Runs for 10-Year Return Period

Table E.7
Applicability of Minimum Discharge Criteria to Ten Test Sites

Site No	Post-Development Land Use	Site Area (acres)	Site Drainage Area (acres)	Pre-dev Model Estimates (cfs) by Ordinance for Return Periods (years)			Minimum Discharge Criteria (cfs) by Ordinance for Return Periods (years)			Minimum Discharge Applicable for Return Periods (years)			Original 1-year Pre-Dev Estimate	Minimum Greater Than Original
				1	2	10	1	2	10	1	2	10		
1	Commercial Small	1.1	2.1	0.2	0.4	1.5	0.2	0.5	1.2		YES		0.5	
2	Commercial Small	3.02	5.21	0.4	0.8	3.0	0.3	0.6	1.5				0.9	
3	Institutional	230	230	6.9	9.5	38.6	4.3	7.3	21.7				na	na
4	Commercial Large	87	62.6	1.1	2.5	10.8	1.3	2.3	6.6	YES			0.1	YES
5	Residential High Density	16.4	41.7	1.0	2.1	8.9	1.0	1.7	4.8				1.4	
6	Residential Single Family	34.38	34.38	1.9	3.6	12.3	0.8	1.4	4.1				3.3	
7	Light Industrial	8.1	5.504	0.1	0.3	1.4	0.3	0.6	1.5	YES	YES	YES	2.4	
8	Residential Mobile Home	23.3	56.64	1.2	2.8	12.4	1.2	2.1	6.1				11.8	
9	Gas Station	5.83	0.96	1.0	1.3	2.2	0.2	0.4	1.1				1.8	
10	Parking Lot	0.96	5.83	0.8	1.6	4.7	0.3	0.6	1.5				1.5	

Census data are not directly available for the Spring Creek drainage basin; therefore, data were collected for both Centre County and State College for the periods of 1940 to present. The data are plotted on Figure E.28. As can be seen from the figure, the trends are similar for the two areas and therefore should be applicable to the basin as well. In addition, it can be observed that a significant increase in population occurred between the decades of the 1940s to 1960s, and the decades of the 1970s to 1990s. It was assumed that development activity, and the associated increases of impervious areas that came from it, should be highly correlated to the population data. Reviewing the hydrologic data and the census data together, it was decided to split the Axemann gage data set into two independent sets each of 21 years in length. The periods selected were 1942 to 1962, and 1974 to 1994. Comparing the two population data sets, there is almost a doubling of the average population between the two time periods. If a hydrologic change has occurred within the basin due to development, it should be observable between these two data sets. These periods were also selected so that neither, of the three major runoff event years (1936, 1972, 1996), or the drought years of the 1960's (1964 through 1967) would be included to bias the data.

In order, to visually see changes within the two data sets, the Double Mass Analysis was used. This method entails cumulatively ranking both data sets by years, and then plotting the ranked data against each other. Ranking the data cumulatively allows one to see changes that occurred over time. If significant changes occurred, they will show up as a sloped line that deviates from unity. Using this method, the annual precipitation data were plotted (refer to Figure E.29). Comparing the data to the to the unity line it can be seen that the two data sets are very similar. This similarity can also been seen in Table E.8, which shows the raw data used. Additionally from the data it can be seen that the annual precipitation data are similar in average, standard deviation, and range. Therefore, although rainfall intensities or single events were not compared or reviewed, this simple analysis indicates that any differences in runoff should not be directly attributable to differences in precipitation. This assumption is statistically reinforced because each data set represents 21 years of continuous data.

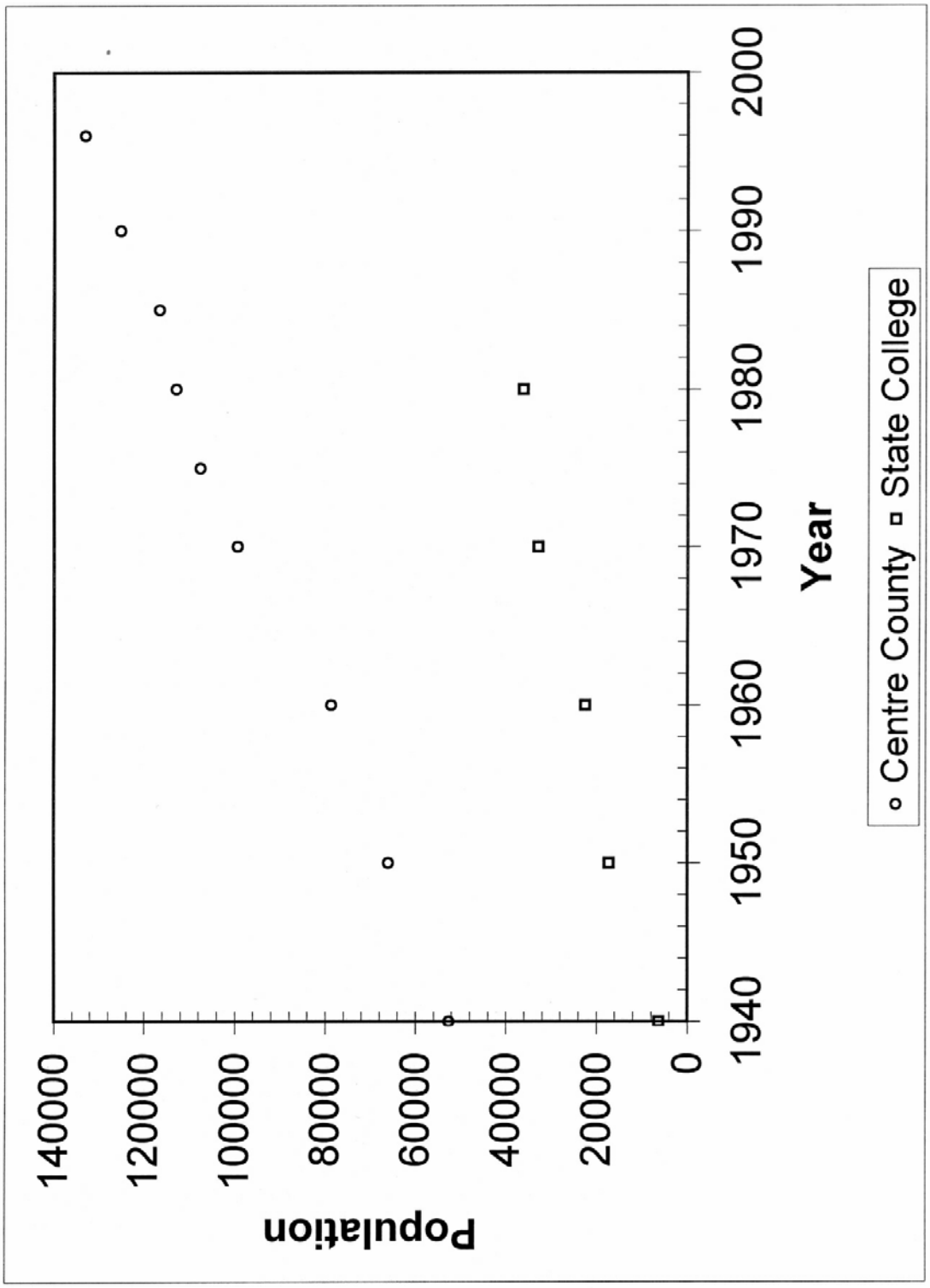


Figure E.28
Census Data for Centre Region Area from 1940 to 1996

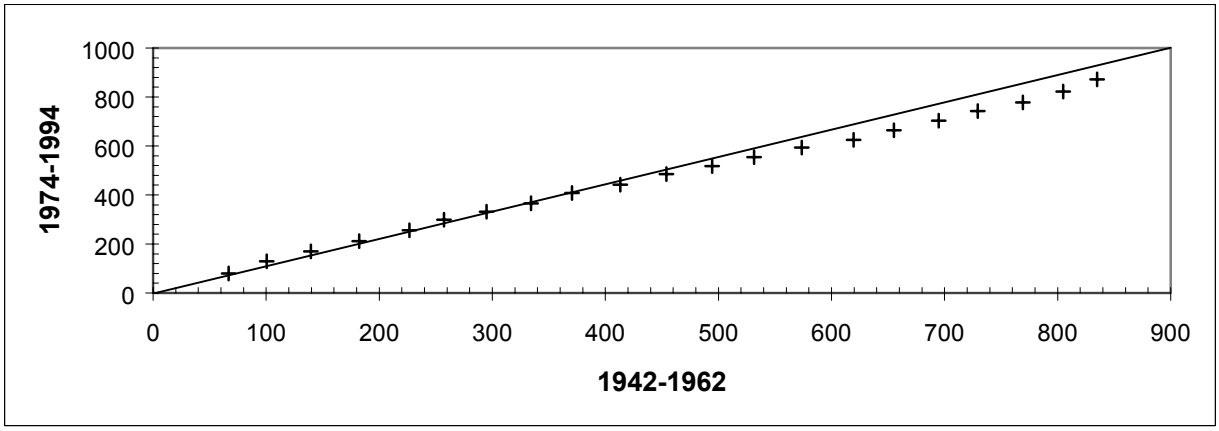


Figure E.29
Double Mass Analysis of Annual Precipitation in the Spring Creek Basin

Reviewing the data plots for annual runoff in inches and peak runoff rate (using an annual exceedance series) in Figures E-30 and E-31, it can be observed that both the annual runoff at the Axemann gage and the peak runoff rates increased due to development. These trends are as would be expected showing that development increases the total runoff volume and the peak rates. However, reviewing the double mass plot of the percentage of the total annual runoff that was baseflow in Figure E.32, it appears that baseflow on an annual basis has not changed within the basin. Unfortunately, viewing the data in this regard on an annual basis would incorrectly lead one to believe that development has not affected baseflow within Spring Creek. However, this oversimplification is not valid. This is because low flows represent only a small portion of total runoff on an annual basis.

Another method used to assess the impact of development on baseflow was to assess the Axemann gage average daily runoff rates provided by the USGS. Although this method does not allow direct separation of the stormflow from baseflow, it provides distinct trends. In order to make the data more visual, the daily average runoff rate was plotted against the day of record (21 years equal 7,669 days) for only average runoff rates less than 100 cfs. Referring to Figures E.33 and E.34, one can observe that during dry periods of the year (typically June to September), the base runoff rates have dropped approximately 5 to 10 cfs at the lowest flows.

Table E.8

Axemann Water Balance (in) in Water Years for Two Time Periods

Year	P (in)	Total Q (in)	Baseflow (in)	Peaks* (cfs)	Year	P (in)	Total Q (in)	Baseflow (in)	Peaks* (cfs)
1942	38.7	12.78	10.68	990	1974	39.88	15.9	13.44	878
1943	33.68	15.46	12.99	712	1975	49.9	17.57	14.54	892
1944	39.11	13.3	11.11	822	1976	39.68	17.14	14.93	791
1945	42.63	13.57	11.73	712	1977	41.94	16.31	13.75	765
1946	44.38	17.56	15.12	685	1978	45.13	25.12	20.32	702
1947	30.58	11.88	11.05	1670	1979	42.13	19.57	15.67	690
1948	37.74	13.92	11.59	671	1980	33.44	20.19	17.53	1990
1949	39.23	13.69	10.91	950	1981	34.03	12.34	10.56	1690
1950	36.19	12.72	11.02	832	1982	40.96	16.39	14.24	1400
1951	42.77	20.91	17.45	696	1983	34.01	14.65	12.62	1140
1952	40.92	15.21	13.22	1430	1984	43.83	20.91	17.01	1740
1953	40.3	16.57	14.31	750	1985	32.77	14.11	12.62	1470
1954	37.17	11.44	10.17	750	1986	36.99	15.85	13.63	1770
1955	42.11	12.73	11.22	920	1987	38.67	14.45	12.81	1020
1956	46	16.97	13.86	1510	1988	31.47	11.43	10.04	1090
1957	35.61	16.34	14.34	1220	1989	39.09	14.54	12.55	1080
1958	39.64	13.4	11.58	802	1990	39.43	12.86	11.35	1070
1959	34.61	10.46	8.65	890	1991	38.62	17.26	15	1180
1960	39.78	14.29	12.5	980	1993	35.59	10.71	9.44	930
1961	35.78	12.34	10.75	802	1993	43.93	19.33	14.91	696
1962	29.97	12.31	9.94		1994	49.6	20.38	16.78	
ave	**38.4**	**14.2**	**12.1**	**939.7**	ave	**39.6**	**16.5**	**14.0**	**1149.2**
stddev	**4.2**	**2.4**	**2.0**	**292.0**	stddev	**5.1**	**3.5**	**2.6**	**398.1**
min	**30.0**	**10.5**	**8.7**	**671.0**	min	**31.5**	**10.7**	**9.4**	**690.0**
max	**46.0**	**20.9**	**17.5**	**1670.0**	max	**49.9**	**25.1**	**20.3**	**1990.0**

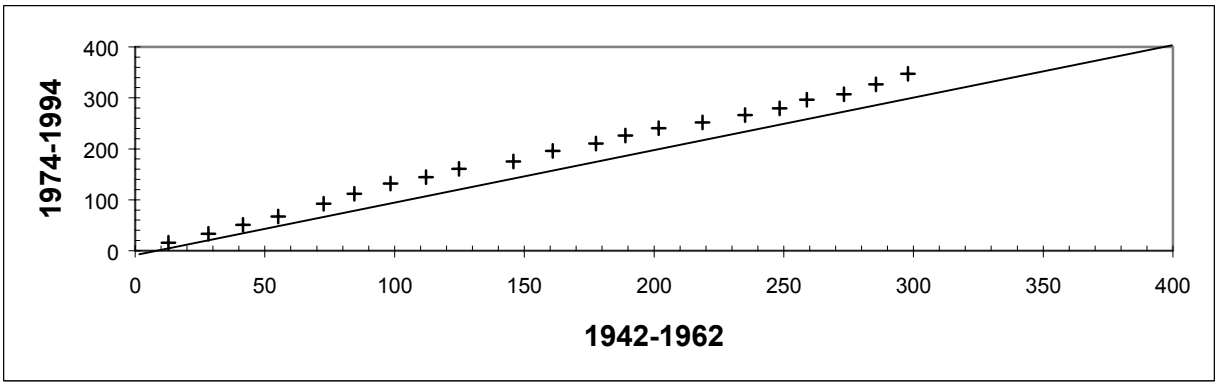


Figure E.30
Double Mass Analysis of Annual Runoff (in) for Spring Creek Basin

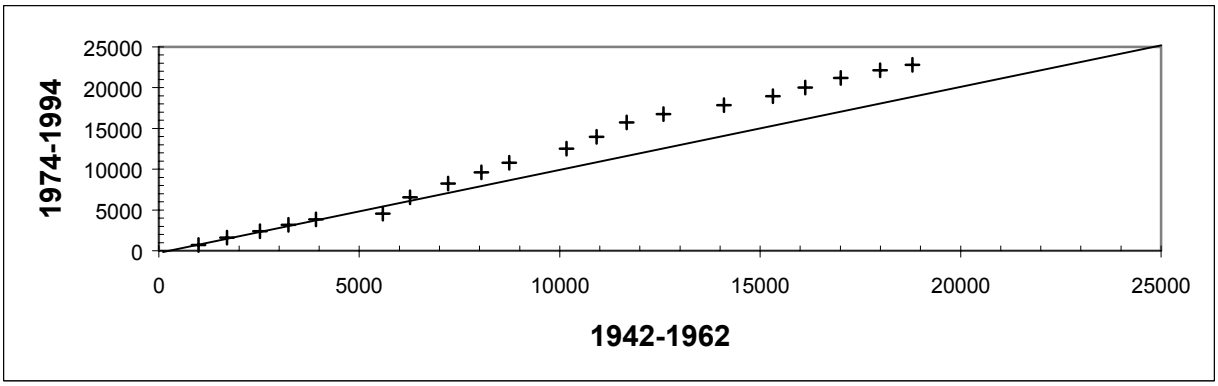


Figure E.31
Double Mass Analysis of Partial Series Peak Runoff Rates (cfs) for Spring Creek Basin

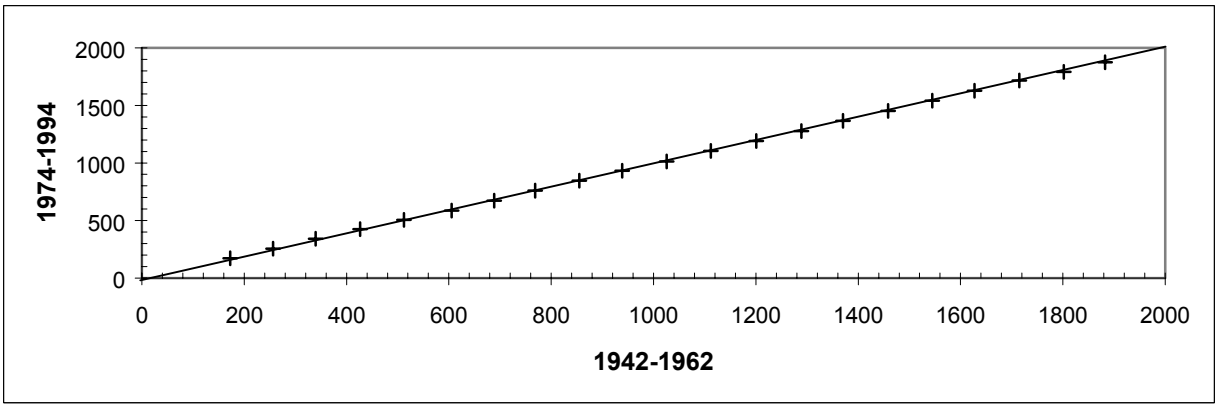


Figure E.32
Double Mass Analysis of Percent of Baseflow for Spring Creek Basin

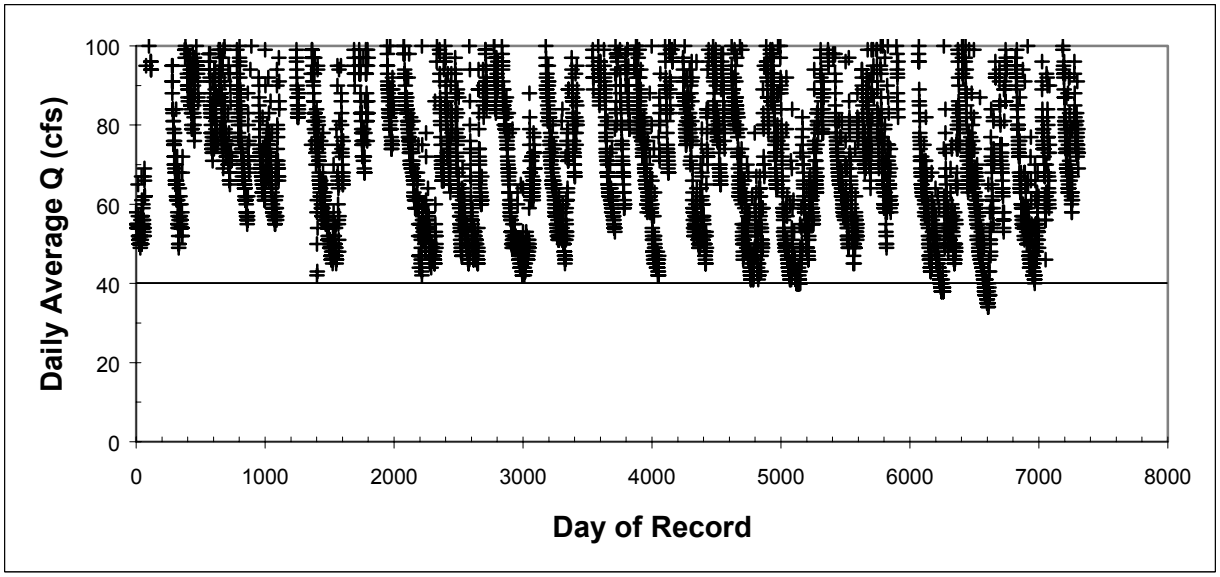


Figure E.33
Daily Runoff for 21-Year Period (1942 through 1962) Showing Annual Minimum Low Flows in Spring Creek

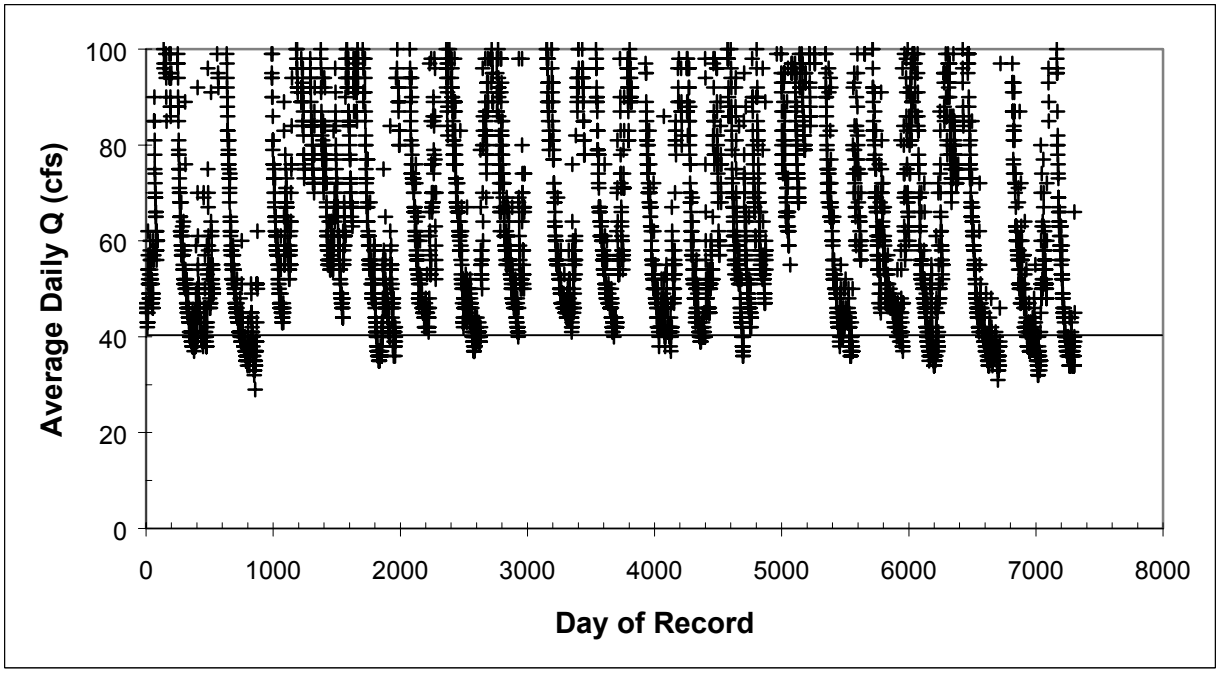


Figure E.34
Daily Runoff for 21-Year Period (1974 through 1994) Showing Annual Minimum Low Flows in Spring Creek

This is equivalent to approximately 25 percent of the flow in Spring Creek during dry periods. Because of this loss, it is critical that recharge be incorporated into a basin wide approach.

Another reason for the differences between Figures E.32, E.33, and E.34 is that as the stream discharges increase due to development, the percentage of that discharge that is considered baseflow remains the same because of the diffuse and conduit flow in the underlying carbonate geology. The typical development practice is to channelize stormwater flows and to connect these discharges into the major drainage ways, which are not able to enter the long term groundwater aquifers, but rather infiltrate into the influent channels and is conveyed through the larger, and quicker, carbonate conduits. Because this conduit flow of the stormwater runoff is still much slower than surface runoff, it appears as baseflow on an annual basis. However, because the runoff has entered the subsurface conduits that still feed the major streams, it has not replenished the groundwater aquifers, which are so important at maintaining the dry period baseflows. Consumptive use of the groundwater by the ever-increasing population expansion increases the stress on baseflow further.

Unfortunately, forcing every site to recharge 100 percent of the site runoff for some design event could accelerate the development of sinkholes and is not practicable. Additionally for some sites, such as gas stations or other critical pollutant source areas, we may not want to encourage recharge at the site due to the high risk of groundwater contamination. In order to come to some reasonable compromise, a philosophy requiring an equivalent natural recharge of 50% of every site was adopted. This will ensure that large impervious areas still provide some measure of recharge. Using this basic understanding, the following computational methods and criteria were developed for recharge.

Runoff from 50 percent of the site must be considered recharged for a 24-hour precipitation depth of 1.5 inches. Using the NRCS's soil cover complex method as a basis, by definition, this would occur for any site that had 50 percent of the total area consisting of a pervious land use equivalent to a weighted CN of 57 (for CN = 57, S = 1.51 in for ARC II). However, for simplicity, this shall be considered accomplished by default for any site with less than or equal to 50 percent impervious area, where the pervious 50 percent of the site consists of a land use of meadow, pasture, grass, or woodland (previously undisturbed areas). For a site that consists of 70 percent impervious areas, at least 20 percent of the site must be directed to a specific

recharge BMP. The volume of runoff that will need to be controlled in a recharge BMP will be computed based on a recharge depth of 1.5 inches for the site impervious areas in excess of 50 percent of the total site area.

Recharge BMPs shall not be used and there are no recharge requirements for Water Quality Sensitive (WQS) developments. Water Quality Sensitive (WQS) developments are defined as land development projects that have a high potential to cause catastrophic loss to local water quality and could potentially threaten ground water reservoirs. The following is a provisional list of water quality sensitive developments that can be added to over time:

- Vehicle fueling stations
- Industrial manufacturing sites
- Salvage yards
- Recycling centers
- Fleet storage areas
- Hazardous material storage areas
- Interstate highways

For the purposes of recharge calculations, lined stormwater management ponds are to be considered impervious. Unlined ponds can be treated as pervious lawn area.

Recharge is done best by leaving undisturbed areas intact over which sheet flow runoff from small areas can be dispersed.

One of the best ways to promote recharge (and gain some water quality benefit) is by removing curbs and gutters from roadways and collecting road drainage in roadside swales. In the past, this practice has been highly unpopular with local municipalities. The reason municipalities want curbs and gutters installed, maintenance issues aside, is that when roadways are poorly designed or constructed, the lack of curbs and gutters can create numerous flooding conditions on residents' properties. Municipalities are often inundated with complaints of flooding, caused by roadway construction, or the erosion in constructed roadway swales. Often times these complaints can be directly attributable to the adjacent property owners who have placed undersized culverts at driveways, or filled in minor drainage ways, or in the case of erosion, poor design or construction. However, several complaints are directly attributable to not considering the actual hydrologic processes that occur within the site.

Unfortunately, hydrologic processes cannot be explained in this simple text, and only a few designers or engineers conducting, or reviewing stormwater management plans are even faintly aware of the actual processes that may occur at a site, but are even more at a loss of how to design for them. Therefore all that can be said regarding the use of curbs and gutters, is that there are times when they should and should not be used. This also can occur within the same development or along the same stretch of road. Requiring them due to poor design or construction practices should not be an issue.

Finally, because this analysis is concerned with trying to adequately represent real processes that occur within the watershed, **there will be areas that cannot physically recharge water**. Some of these areas are exfiltration areas, commonly found at the base of the wooded hillsides. These areas respond more closely to saturation excess processes such as drainage ways near the major stream or in flood plains. These areas should always be exempt from recharge criteria. However, the site designer should consider other possible runoff problems that may occur in these areas.

Recharge Standard Change in Response to Revised State SWM Policy

The recharge standard presented in the previous section intentionally exempted sites having less than a 50 percent impervious area ratio from the requirement to compute and mitigate a specified recharge volume. However, in the fall of 2001, the State released a revised stormwater management policy which stated that sites in high quality and exceptional value watersheds could not be exempted from recharge requirements. In response to this change in policy, an alternate standard was devised which required that all sites provide recharge mitigation while maintaining sensitivity to the karst (sinkhole prone) nature of the watershed. This standard was designed to require the same recharge mitigation volume as the original standard for highly impervious sites. The revised standard requires that a recharge volume equal to 0. 5 inches of runoff from all impervious surfaces be mitigated on all sites. To encourage low-impact development the following credits can be applied against the required recharge volume:

- Residential and commercial buildings with roof areas less than 5,000 square feet can be removed from the computed impervious area when these roof areas are sumped to dry wells.

- All or portions of driveways, roadways, and parking areas can be removed from the impervious area calculations when sheet flow from these areas is directed to undisturbed natural buffer/filter areas.
- Sidewalks separated from roadways and / or other impervious surfaces by a grass strip of equal or greater width than the sidewalk itself can be removed from the impervious area calculation when these areas are graded so that sheet flow is directed to the grass strip.
- Impervious areas tributary to natural closed depressions can be subtracted from the total site impervious area as long as a qualified geotechnical engineer or soil scientist certifies to the soundness of these site-specific applications.
- Impervious areas tributary to man-made closed depressions can be subtracted from the total site impervious area as long as a qualified geotechnical engineer or soil scientist certifies to the soundness of these site-specific applications.
- The entire capture volume provided in a pond without a subsurface drain may be used as a credit towards the recharge volume requirement.
- Fifty percent (50%) of the capture volume in a pond that includes a subsurface drain may be credited towards the recharge volume requirement.

After credits, the remaining recharge volume must be directed to a recharge BMP such as infiltration trenches, beds, etc. This revised recharge requirement provides a consistent recharge standard while maintaining sensitivity to sinkhole development, and acknowledging natural recharge associated with the disconnection of impervious areas and other low impact development techniques.

Pond Capture Volume

Recent research (Fennessey et al., 2001) has shown that even when stormwater management ponds are used, a radical change in the hydrologic response of small watersheds occurs following development. These changes occur even when the pre-development peak runoff rate estimates are accurately computed for a watershed. The change that occurs is in the frequency of small pond peak rate discharges for storms less than the 2-year return period. The increase in frequency of these minor flows is related to increased nuisance flooding, and can significantly alter or accelerate stream morphology.

Pond capture volume is specifically intended to mitigate increased frequency of nuisance flooding in areas where detention facility. This criteria, is only related to surface stormwater management ponds, and therefore the areas tributary to them. Subsurface stormwater management piping systems or vaults that connect directly to drainpipes should not require a capture volume to be used due to the hardships caused.

The runoff capture volume will be equivalent to a runoff depth of 0.25 inches from all impervious areas tributary to the stormwater facility. After capture in the stormwater facility, this runoff volume will be allowed to infiltrate, evaporate or dewater from a subsurface drain system connected directly to the pond's principal spillway. The ponded water should be designed to dissipate within 48 hours. No principal spillway orifice (except those connected to subsurface drains), regardless of how small, shall be below the pond elevation equivalent to this volume.

This depth of runoff capture will help ensure that minor precipitation events do not create a nuisance flooding problem downstream due to development of large impervious areas. The purpose of the capture volume is to control peak runoff rates from impervious areas and is not related to water quality. However, ponds designed with water quality volumes equal to or greater than the required capture volume are considered to have met the capture requirement by default.

Stream Bank Morphology

Stream bank morphology is related to the development of natural channel forms including channel cross sectional shape, stream down-valley slope, and stream sinuosity (degree of meandering). As indicated above, recent research (Fennessey et al., 2001) has shown that releases from stormwater management facilities change the frequency of peak rate discharges for storms less than the 2-year frequency event. The increase in frequency of these minor flows can significantly alter or accelerate stream morphology.

There is a large degree of confusion and disagreement in regards to which storm event can realistically be considered the most influential in forming a stable stream. However, following uncontrolled development, it is known that, both stream discharge magnitudes and frequency increase. These discharge changes can cause both the downstream and upstream sections to be radically altered. In order to make the subject clearer, the following information is provided.

1. A single major runoff event can radically alter both the stream profiles and sections in a single event more than the erosion from a large period of years of normal flows. Therefore, the stability of any channel section for design events from 10 to 100+ years should not be considered except in regards to safety issues such as stability or scour at stream crossings.

2. Bank full discharge, typically associated with the mean annual flood of a reach, and the 1-year, 24-hour runoff event, which is often computed for stormwater management purposes, are both larger than the accepted stable channel discharge.

3. The discharge that most river morphologists would concur forms a channel is referred to as the dominant discharge. Work by Nixon (1959), and Wolman and Leopold (1957) found that this discharge could be related to a return period of between 6 and 9 months.

4. Small developing sites do not have base flow or, in most instances, defined channels or point outlets within the site. Therefore, the problems of site channel or swale stability are more often a consideration of erosion caused by poor designs, construction practices, or unknown site conditions and should not be considered due to normal channel forming processes.

5. The over estimation of pre-development runoff rates has a significant negative impact on Streambank morphology.

6. If no capture depths in ponds are used, the radical change in the frequency of small (< 2 year) runoff events has a significant negative impact on streambank morphology.

Nonetheless, in a natural or well-designed channel, changes in channel form and shape may occur due to the frequency of the dominant discharge from stormwater management ponds that discharge controlled runoff from large impervious areas.

However, for the Spring Creek Basin, the use of: 1) capture volumes in ponds; 2) the use of both the 1- and 2- year return periods for peak rate analysis; and 3) the fact that the

overestimation of pre-development runoff rates was accounted for are considered satisfactory for controlling negative morphology changes.

Water Quality Volume

Maintaining the health and quality of both surface water and ground water resources within the Spring Creek Watershed is a high priority of the regions residents. Because of the carbonate geologic structure underlying the Spring Creek Drainage Basin, land development activities within the watershed have the potential to have significant and direct impacts not only on surface waters, but also on the regions high quality groundwater aquifer. There is strong regional support for implementation of best management practices (BMP's) for stormwater quality control within the watershed. The Pennsylvania Handbook for Best Management Practices (PHBMP) provides design guidance for numerous BMP's which could be applied. However, if BMP's are applied without considering the actual hydrologic processes occurring within the basin, or limitations in karst environments, their implementation may result in a greater potential for pollution. Therefore, prior to simply identifying what BMPs should be used at developing sites or how water quality methods should be computationally addressed, a review of traditional water quality considerations, and how the carbonate geology of the basin and the site affects these considerations, will be made.

Development activities can have a profound influence on the quality of water within a drainage basin. This is because development dramatically alters the local hydrologic cycle. The hydrologic changes start during the initial clearing and grading that occur during construction. Vegetative cover that had previously intercepted precipitation is largely removed, and natural closed depressions that had temporarily ponded water are generally graded to a uniform slope. The spongy humus layer of the forest floors that had absorbed rainfall is scraped off, eroded or severely compacted. The filtering process that naturally occurred through these natural ground covers and topographic features is lost as a result of the land development activity. Having lost portions of its natural storage capacity and the land's ability to retard and filter storm flows, a cleared, graded and developed site produces a greater percentage of stormwater runoff and significantly greater pollutant loads. Unfortunately, stormwater management facility designs currently employed in the land development industry do not adequately address stormwater quality issues.

Stormwater quality impacts resulting from development activities are common to all watersheds regardless of regional geology. Development increases the impervious surfaces on a site, which accumulate pollutants deposited from the atmosphere (dry deposition), or blown in from adjacent areas. Additionally, vehicles are a significant source of pollutants such as hydrocarbons from oil leaks and metals from brake pad dust. Most of these pollutants remain on the impervious surfaces until a precipitation event, at which time; the pollutants quickly wash off and can be rapidly delivered to downstream surface waters and/or groundwater. In addition to developed areas, agricultural practices or woodland management practices can also have a detrimental effect on water quality, especially in relation to nutrient management, herbicide, or pesticide use.

Some of the more common pollutants found in urban stormwater runoff can be categorized as: nutrients, sediments, pathogens, organic enrichment, toxic pollutants, and salts.

- ***Nutrients*:** Urban runoff can have elevated concentrations of both phosphorus and nitrogen, which can enrich streams, lakes, reservoirs and estuaries (known as eutrophication). Excess nutrients promote algal growth that blocks sunlight from reaching underwater grasses and depletes oxygen in bottom waters. Excess nutrients from sites are generally related to lawn maintenance or fertilization activities.

- ***Sediments*:** Sources of sediment include washoff of particles that are deposited on impervious surfaces and the erosion of streambanks and construction sites. Both suspended and deposited sediments can have adverse effects on aquatic life in streams, lakes and estuaries. Sediments also transport other attached pollutants. Suspended solids during construction activities are controlled by temporary construction facilities. Permanent ground cover, stable channel designs, and site maintenance plans are an important part in limiting suspended solids in runoff from a site following development.

- ***Pathogens*:** Pathogens are bacteria, protozoa, and viruses that can cause diseases in humans. Bacteria levels in stormwater runoff routinely exceed public health standards for water contact recreation. Stormwater runoff may increase the cost of treating drinking water at water supply reservoirs. Pathogens in stormwater runoff are generally contributable to animal waste or improperly functioning or overloaded sanitary facilities. Pathogens generally cause water quality degradation in slow-moving waterways.

- ***Organic Enrichment*:** Organic matter, washed from impervious surfaces during storms, can present a problem in slower moving downstream waters. As organic matter decomposes, it can deplete dissolved oxygen in streams. Low levels of oxygen in the water can have an adverse impact on aquatic life.

- ***Toxic Pollutants*:** These commonly include hydrocarbons, pesticides, herbicides, and trace metals. Vehicles leak oil and grease, which contain a wide array of hydrocarbon

compounds, some of which can be toxic at low concentrations to aquatic life. Cadmium, copper, lead and zinc are routinely found in stormwater runoff. These metals can be toxic to aquatic life at certain concentrations and can also accumulate in the sediments of streams. A modest number of currently used and recently banned insecticides and herbicides have been detected in urban streamflow at concentrations that approach or exceed toxicity thresholds for aquatic life. The uptake and removal of toxic pollutants is an issue that can be directly addressed in most site designs.

- ***Salts*:** Salts that are applied to roads and parking lots in the winter months appear in stormwater runoff and meltwater at much higher concentrations than many freshwater organisms can tolerate. Road salts used on a site could impact groundwater resources.

In addition to the six defined categories, thermal impacts and trash can cause surface water quality impacts.

- *Thermal Impacts.* Impervious surfaces may increase temperature in receiving waters, adversely impacting aquatic life that requires cold and cool water conditions (e.g., trout). This is particularly important, since the many portions of Spring Creek are considered cold water fisheries. Thermal impacts can be best addressed within the watershed by maintaining riparian zones and especially leaving wooded land uses around drainage ways.

- *Trash and Debris.* Considerable quantities of trash and debris are washed through the storm drain networks. The trash and debris accumulate in streams and lakes and detract from their natural beauty. Trash and debris must be addressed by maintenance at each site.

The slow movement of water through the soil layers is essential for filtering stormwater runoff and also for replenishing groundwater reservoirs. However, if stormwater runoff is directed into the soil without adequate treatment, development activities can also degrade groundwater quality. Unfortunately, the carbonate geology of the drainage basin can often provide direct conduits between these pollutants and ground water reservoirs. If not adequately designed, BMPs can result in the creation of sinkholes, which can become direct connections or conduits, between storm runoff and ground water reservoirs. Additionally, streams within the basin, such as the Big Hollow, are known to have many existing sinks along their length that can serve as direct conduits to groundwater reservoirs and under most periods of the year are dry channels for a majority of their length.

The most common method used for establishing water quality requirements for BMPs is through the concept of a capture depth or water quality depth. Unfortunately, because of the popularity of the acronym “BMP”, numerous misunderstanding and over-exaggerations have occurred. A plethora of manuals exist and are being developed that praise the benefits of water quality

BMPs, many even noting effectiveness levels or relative ranks between the different methods. This has also been done to a degree in this document**. But in actuality, the best way to promote water quality within a drainage basin is by leaving drainage ways undisturbed, disconnecting stormwater runoff systems, and limiting impervious areas.**
Prior to establishing, a water quality criterion for the Spring Creek drainage basin, a review of existing data and how the issue has been addressed in other watersheds was made.

The effectiveness of BMPs for water quality enhancement as reported in general literature is based on extremely limited data or in some cases, none at all. The EPA funded an extensive water quality monitoring study known as the National Urban Runoff Program (NURP) in the 1970s and early 1980s that is used as a basis for determining the effects of land development on water quality. However, development site BMPs were not specifically addressed in the original NURP study and the use of these BMPs has only occurred in the last 15 to 20 years. Using the simple conceptual philosophy of extending the residence time of stormwater runoff, filtering where possible, and allowing plant uptake of the pollutants in certain wetland areas is the basic premise that has been used for determining the effectiveness or benefits for many of these BMPs.

The basis for many water quality designs was developed after the work done by Roesner and others in the late 1980s and early 1990s. Using computer simulations with hypothetical stormwater management ponds, Roesner et al. (1991) showed that in Cincinnati Ohio, capturing the first 0.25 inches of runoff resulted in capturing 75 percent of the annual runoff. When the capture depth was increased to 0.5 inches, it resulted in capturing 93 percent of the annual runoff. Other work has shown that trying to capture over 90 percent of the annual runoff in ponds requires excessive increases in pond storage volumes with little gain.

Because of the way pollutants are washed off of impervious surfaces during the initial stages of large events or during small precipitation events, it can be assumed that capturing the small events, more frequently will allow for the filtering of pollutants from stormwater. Therefore, several States have adopted first flush criteria for stormwater runoff from impervious surfaces that relates to the first 0.25 to 0.5 inches of runoff from impervious surfaces. Impervious surfaces are only considered because most pervious areas will not produce runoff for

precipitation events between 0.25 and 0.5 inches. Unfortunately, here is where a large degree of confusion has occurred.

States such as Maryland have adopted water quality capture volumes with precipitation functions included (MD DOE, 1997). Although the methodology adopted by Maryland' s DEP is very valid and technically correct, others have incorrectly assumed that water quality therefore should be based on precipitation depth probabilities for regional areas. This assumption is not supported by data.

There are several acceptable ways to statistically analyze precipitation data for use in developing water quality equations. Precipitation data for State College were used for an 11-year period from 1986 to 1996 (1996 was the precipitation year of record for the area). In order to remain consistent with the 24-hour partial series precipitation depths commonly used for modeling purposes, the daily data were converted to 24-hour data by a method similar to Hershfield (TP-40, 1961). The data then were evaluated using the recommended method in the Pennsylvania Handbook of Best Management Practices for Developing Areas (PHBMP) (PA DEP, 1998), and three additional methods. Two distinct differences in the methods used are that one is based on a percentage of the total precipitation received during the year, and the other is based on the number of times events of a certain total depth occurred within the year. Considering water quality, using the number of times events of a certain total depth occurred within the year is more applicable to the first flush concept.

The results of the historical data analysis were also compared to what the PHBMP states are the percent probabilities using the PaDEP IDF curve data for Regions 2 and 3. The results are presented in Table E.9. Reviewing Table E.9, a compromise was made that would protect water quality and minimize the potential to induce sinkhole formation in regions underlain by carbonate geology.

Table E.9

Historical Precipitation for Spring Creek by Different Analyses Methods

PHBMP Manual Data			
	% Event Volumes		
	60%	75%	90%
Region 2	0.59	0.88	1.48
Region 3	0.63	0.97	1.6
Spring Creek Data by PHBMP Manual Method			
	% Event Volumes		
	60%	75%	90%
State College	0.73	1.06	1.75

Spring Creek % Events Fully Captured by First Flush		
Depth	All Events	Only P>0.1 inch
0.25"	74%	37%
0.5"	87%	67%
0.61"	90%	76%
1.01"	96%	90%

Using 24-hour converted data

Event depth method is based on % of total annual precipitation volume

It was decided to develop an equation that would have the following properties. At a minimum, the capture depth would be equal to 0.25 inches. This depth would represent full capture of approximately 74 percent of all precipitation events or 37 percent of the precipitation events only greater than 0.1 inches in total depth. Additionally, it was desired to fully capture 0.5 inches of runoff for all precipitation events greater than 0.1 inches in total depth for sites that had 65 percent impervious area. Sixty-five percent was selected because it represents an average of the allowable maximum impervious coverage for commercial developments by most municipal zoning ordinances within the basin. Another feature of the equation is that it would require approximately 1.5 inches to be captured if a site were 100% impervious. This would represent full capture of approximately 98 percent of the precipitation events in the basin (note: this is a different statistic than the 90% of the annual runoff volume). The 1.5-inch value also coincides with the depth of rainfall required prior to runoff occurring from most pervious areas within the

basin. (Naturally, the capture depth should not be enforced for all sites, particularly those that were previously 100 percent or nearly 100 percent impervious.) Finally, the equation should produce minor changes in the required capture depth for sites with low percentages of impervious area and increase markedly for highly impervious sites.

Additionally, It was decided to establish two different general criteria for water quality volumes. The first would be for non-sensitive areas underlain by carbonate geology. The volume criteria established for these areas, is less (for impervious areas up to 65 percent), because of the desire to minimize the formation of sinkholes. The second criteria would apply to the non-carbonate areas, or any water quality sensitive development or areas within sensitive groundwater areas regardless of the underlying geology. In these areas, the risks associated with the additional volumes are less than the risk of groundwater contamination.

The resulting water quality equations that were developed for use in the non-sensitive areas of Spring Creek Basin that are underlain by carbonate geology are:

$$WQ_{depth} = 0.25+(0.012)2.9^{[0.044(SIA)]})$$

$$WQ_v = WQ_{depth}(A)/12$$

Where: WQ_v = water quality volume in acre-feet
WQ_{depth} = depth in inches that must be captured for impervious areas
SIA = percent of site impervious area (all paved areas and roof with asphalt based roofs)
A = total of all paved areas and roof with asphalt based roofs on site in acres

The required water quality volume that must be treated for any water quality sensitive (WQS) development or on sites in sensitive areas underlain by carbonate rock, and all areas not underlain by carbonate rock is to be computed within the entire Spring Creek Basin as:

$$WQ_{depth} = \text{the larger of 0.5 inches or } 0.25+(0.012)2.9^{[0.044(SIA)]}$$

$$WQ_v = WQ_{depth}(A)/12$$

Where: WQ_v = water quality volume in acre-feet
WQ_{depth} = depth in inches that must be captured for impervious areas
SIA = percent of site impervious area (all paved areas and roof with asphalt based roofs)
A = total of all paved areas and roof with asphalt based roofs on site in acres

The water quality volume must be captured and filtered through a water quality BMP.

These sets of equations weight an acceptable level of control for water quality with the risk of causing groundwater contamination. How these equations compare to those used in the State of Maryland can be seen in Figure E.35 and Table E.10. There is a degree of similarity between the equation developed and those used in the State of Maryland; the major difference being in the higher percentages of site impervious area. This difference is primarily due to the rural setting of the Spring Creek Basin, and because the impervious area is one of the leading indicators of watershed health.

Extreme Event Assessment

Design storm selection normally requires that watershed wide modeling be conducted to simulate hydrologic responses from multiple sub-watersheds for a variety of storm events under various rainfall durations. Watershed wide modeling was not included in the base scope of services for this study, and, therefore, could not be used as a basis for release rate or other analysis to assess appropriate design storms for analysis.

However, an analysis was performed to assess the importance of including extreme events (typically the 50-year and 100 year) in traditional stormwater management pre- and post-development calculations. The approach was to simulate future development scenarios within both the Houserville and Shiloh Road sub-watershed models previously developed, and evaluate the increase in runoff produced for a variety of storms including the 1-, 2-, 10-, 25-, 50-, and 100-year events. The Houserville model was used to simulate impacts to a wide basin area if no stormwater controls were used for future development; The Shiloh Road model was then used to assess the application of stormwater controls under a variety of storm sets.

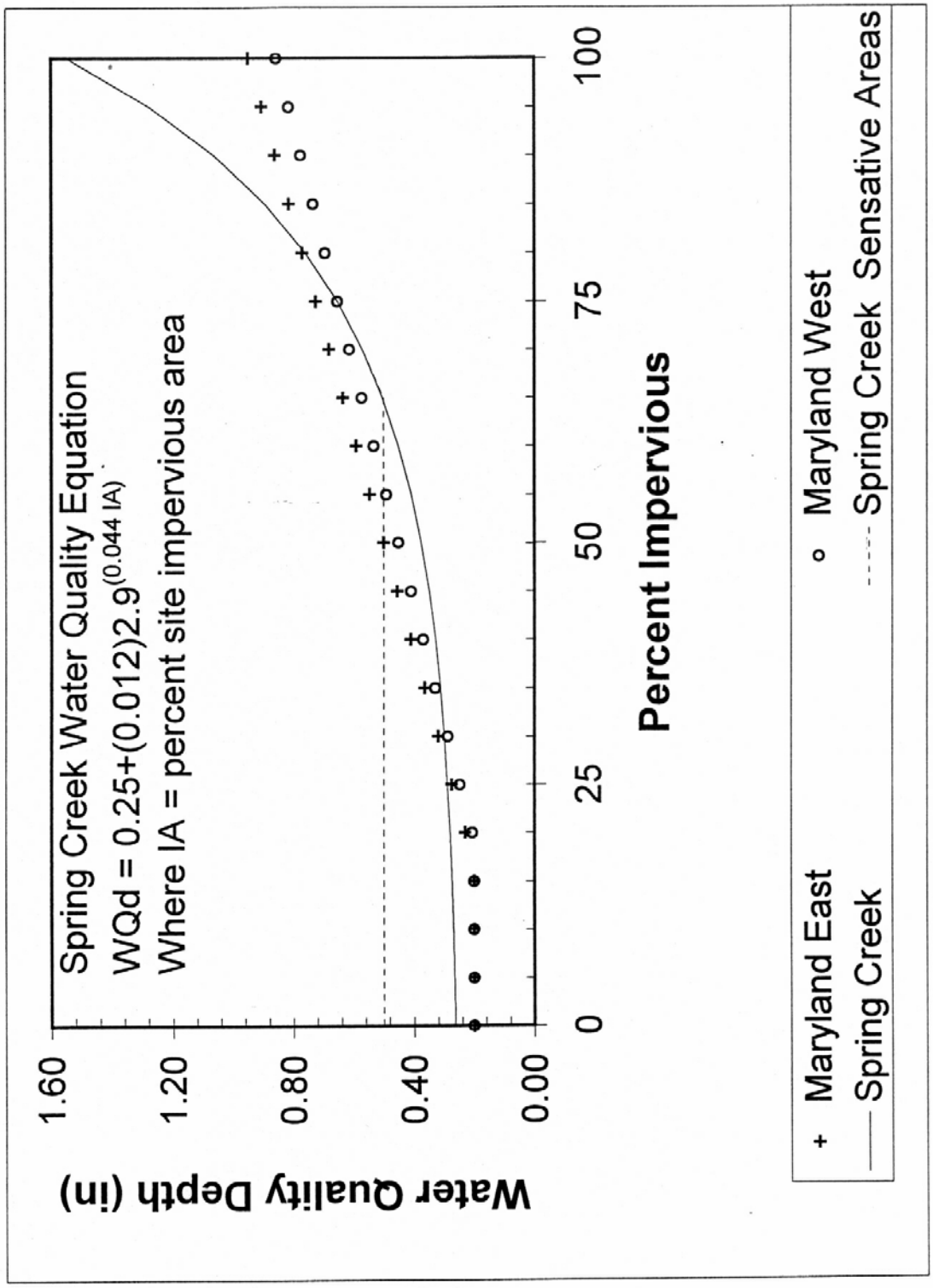


Figure E.35
Comparison of Spring Creek Water Quality Depth Equations Versus Maryland Water Quality Equations

Table E.10

Comparison of Spring Creek and Maryland Water Quality Volume Requirements
[Based on Maryland Stormwater Design Manual Depth One (Pg. 2.2 – 2.7)]

Site Percent Impervious	Eastern Zone Water Quality Depth (Inches)	Western Zone Water Quality Depth (Inches)	Non-Sensitive Area Spring Creek Water Quality Depth (Inches)	Sensitive Area Spring Creek Water Quality Depth (Inches)
0	0.20	0.20	0.26	0.50
5	0.20	0.20	0.27	0.50
10	0.20	0.20	0.27	0.50
15	0.20	0.20	0.27	0.50
20	0.23	0.21	0.28	0.50
25	0.28	0.25	0.29	0.50
30	0.32	0.29	0.30	0.50
35	0.37	0.33	0.31	0.50
40	0.41	0.37	0.33	0.50
45	0.46	0.41	0.35	0.50
50	0.50	0.45	0.37	0.50
55	0.55	0.49	0.41	0.50
60	0.59	0.53	0.45	0.50
65	0.64	0.57	0.50	0.50
70	0.68	0.61	0.57	0.57
75	0.73	0.65	0.65	0.65
80	0.77	0.69	0.76	0.76
85	0.82	0.73	0.89	0.89
90	0.86	0.77	1.06	1.06
95	0.91	0.81	1.28	1.28
100	0.95	0.86	1.55	1.55

Maryland minimum depth for W.Q. volume = 0.2 inches for < 17% impervious area

Recharge volume is considered part of total W.Q. volume

Houserville Sub-Watershed Future Conditions Analysis:

As indicated above, the Houserville sub-watershed model was used to simulate the impacts future development would have on storm events of various frequencies. Projected population growths for the next 10 years were used to adjust for changes in land use and stormwater runoff conveyance systems. These changes, accounted for by an adjustment in the curve number and time of concentration, were than applied to the previously developed model. The model was run for the 1-, 2-, 10-, 25-, 50-, and 100-year events

Peak discharges from each of the model runs are presented in Table E.11 along with historic data and the previous results from the existing conditions model runs. The results indicate that discharges increased from over 31 percent for the 1-year return period to only 9 percent for the 100-year return period. These results confirmed the generally accepted conclusion that, development affects the runoff magnitude from frequent event more significantly than it does for extreme events. For a 100-year return period runoff event, an increase from 6734 to 7331 cfs can be considered negligible, especially since the resulting impact on flood levels could be considered minimal.

Table E.11

Comparison of TR-20 Output for Existing Conditions Versus Future Conditions Using Parameters as Computed by the Proposed Standard for the Subareas within the Houserville Gage Watershed

Return Period (Years)	Peak Runoff Rates (cfs) for Modified Model			Percent Increase in Peaks**
	Historical Data	TR-20 Model for Existing Conditions	TR-20 Model for Future Conditions	
1	600	433	569	31.4
2	690	828	1029	24.3
5	825	1360	1623	19.3
10	2370*	2385	2762	15.8
25		4716	5265	11.6
50	N/A	5707	6291	10.2
100	N/A	6734	7331	8.9

Shiloh Road Sub-watershed Future Conditions Analysis:

The Shiloh Road model was used to assess the application of stormwater controls for a variety of storm events to assist with design storm selection. A unique methodology was developed for analyzing development within the Shiloh Road Watershed.

The first step was to evaluate the pre-development controlling runoff rates within the watershed. This was done using the proposed computational procedures developed earlier in this chapter, as well as the current ordinances computational methods. College Township's Stormwater Management Ordinance, Chapter 175, was used as the existing ordinance. This ordinance was selected for use because most of the Shiloh Road Watershed is located within College Township, and several municipalities have adopted similar ordinance requirements. The original 31-subarea model was used for this analysis. In addition, the peak runoff rates using traditional TR-20 determined parameters were also computed.

Table E.12 presents the results of this initial evaluation. The model incorporating the hydrologic methods proposed earlier in this section is labeled "Shiloh road Base like Model Using 31-Subareas"; the model incorporating existing ordinance provisions is labeled "As Computed by Existing Chapter 175 Ordinance"; and the TR-20 model is labeled as such. The results illustrate that the proposed hydrologic methods result in a dramatic reduction in the pre-development peak runoff magnitude. In addition, it is noted that the current ordinance peak runoff rates are significantly lower than the TR-20 model with traditionally computed parameters; the difference can be attributable to the requirement that all existing conditions land uses be set as meadow or woods in good condition in the current ordinances. However, referring to the table, one observes that the current ordinance, even with these land use criteria, still produce much higher estimates than that of the proposed ordinance methodologies. The difference at low return periods is one of the reasons that incidences of nuisance flooding are increasing throughout the basin.

Table E.12

Shiloh Road Existing Conditions Simulations

TR-20 Model	Return Period (Years)						
	1	2	5	10	25	50	100
Shiloh Road Baseline Model Using 31-Subareas	26	58	107	190	347	495	677
As computed by Existing Chapter 175 Ordinance	78	151	248	402	627	802	1045
TR-20 Model with Traditionally Computerized Parameters	293	429	870	1259	1535	1848	2059

The second step in the analysis was to evaluate runoff from the Shiloh Road Sub-watershed for proposed development conditions. This required that the 31 subarea model be modified to reflect property boundaries instead of true watershed subareas for the portion or the watershed expected to be developed (see Figure E.36 for the developed condition sub-watershed map). Test runs were made with this new developed subareas model (referred to as Dev-Subareas) to make sure there would not be any significant differences between it's results and that of the 31 subarea model. The results are presented in Table E.13. Because of the large property areas used, the runoff rates did not significantly change. This new property based model was then used as the baseline for the all additional future conditions model runs.

Table E.13

Comparison of 31-Subarea Model to the Developed Subarea Model

TR-20 Model	Return Period (Years)						
	1	2	5	10	25	50	100
Shiloh Road Baseline Model Using 31-Subareas	26	58	107	190	347	495	677
Shiloh Road Baseline Model Using Dev-Subareas*	23	55	108	199	374	516	704
Percent Difference 31-subarea to Dev-Subarea Model	13%	5%	-1%	-4%	-7%	-4%	-4%

*Developed subareas based on property lines in conjunction with natural watersheds
Baseline is model based on ordinance developed herein

Using the future development scenario illustrated in Figure E.2, stormwater management ponds were designed for each individual property in the Shiloh Road Watershed using actual stage/storage, and stage/discharge data. Two ponds were developed for each site, one based on the pre-development runoff rates determined by the proposed computation methodologies and the other based on the pre-development runoff rates determined by the current ordinance computation methodologies. The stage/storage was the same for both ponds, while the stage/discharge was modified as required. The watershed model was then run using each of the prospective pond design scenarios.

Figure E.36

11x17

This more rigorous method of determining post-development increases is also actually a more realistic model. The reason is that although runoff estimates have a large degree of uncertainty from natural land areas, once developed, the hydrologic response of watersheds or developments is quite accurate (accepting the precipitation intensity, duration, and frequency). The resulting extended discharge hydrograph, more accurately reflects how changes in combined hydrographs may occur, than simply using the traditional release rate concept.

Separate pond stage/discharge relationships were also developed to evaluate the zero increase in pre- to post-development runoff using the following design storm criteria:

1. The 1 yr, 2-yr , 5-yr, 10-yr, and 25-yr storms for drainage areas less than 10 acres, and the 1 yr, 2-yr , 5-yr, 10-yr, 25-yr, and 50-yr storms for drainage areas greater than 10 acres (the 100-year event was allowed to flow undetained over the emergency spillway); and

2. The 1 yr, 2-yr , 5-yr, 10-yr, 25-yr, 50-yr, and 100-year storms.

An additional model simulation was conducted assuming the same development in the watershed without stormwater controls.

This analysis was conducted to determine if the 100-year return period runoff event, should be kept as a design criteria for ponds. The reasoning behind this is that the majority of stormwater management related problems are related to frequently occurring events, and if the design criteria for the 100-year event could realistically be removed without causing undue harm, more emphasis could be placed on frequently occurring events.

A summary of the results of these model simulations can be seen in Table E.14. It can be observed that providing no additional detention would significantly increase runoff at the outlet of the Shiloh Road Watershed. However, it can also be seen that removing the 100-year design criteria from the proposed ordinance does not significantly increase flows from the Shiloh Road Watershed. In addition, it is noted that peak discharges are significantly reduced below the current ordinance control levels.

Table E.14

Future Conditions Model Estimates Compared

TR-20 Model	Return Period (Years)						
	1	2	5	10	25	50	100
Shiloh Road Baseline Model using Dev-Subareas*	23	55	108	199	374	516	704
Post Dev. No Detention	143	243	367	545	831	1053	1382
Post Dev. Based on Baseline w/25 or 50 year max	23	58	106	201	375	515	800
Post Dev. Based on Baseline w/100 year maximum	23	58	106	201	375	516	687
Post Dev. Based on Chapter 175 w/25 or 50 year maximum	66	129	209	331	543	710	1020
Post Dev. Based on Chapter 175 w/100 year maximum	66	129	209	331	543	713	924

Therefore, it was recommended that the analysis of the 100-year return period event be removed from the required peak runoff rate analysis. The emergency spillway of all stormwater management facilities must be sized to safely pass the 100-year flow, and safe conveyance of this storm event must be accommodated by the land development design. However, due to political influences, the 100-year event was retained as it was felt that we could not convince downstream landowners that no additional harm would be caused.

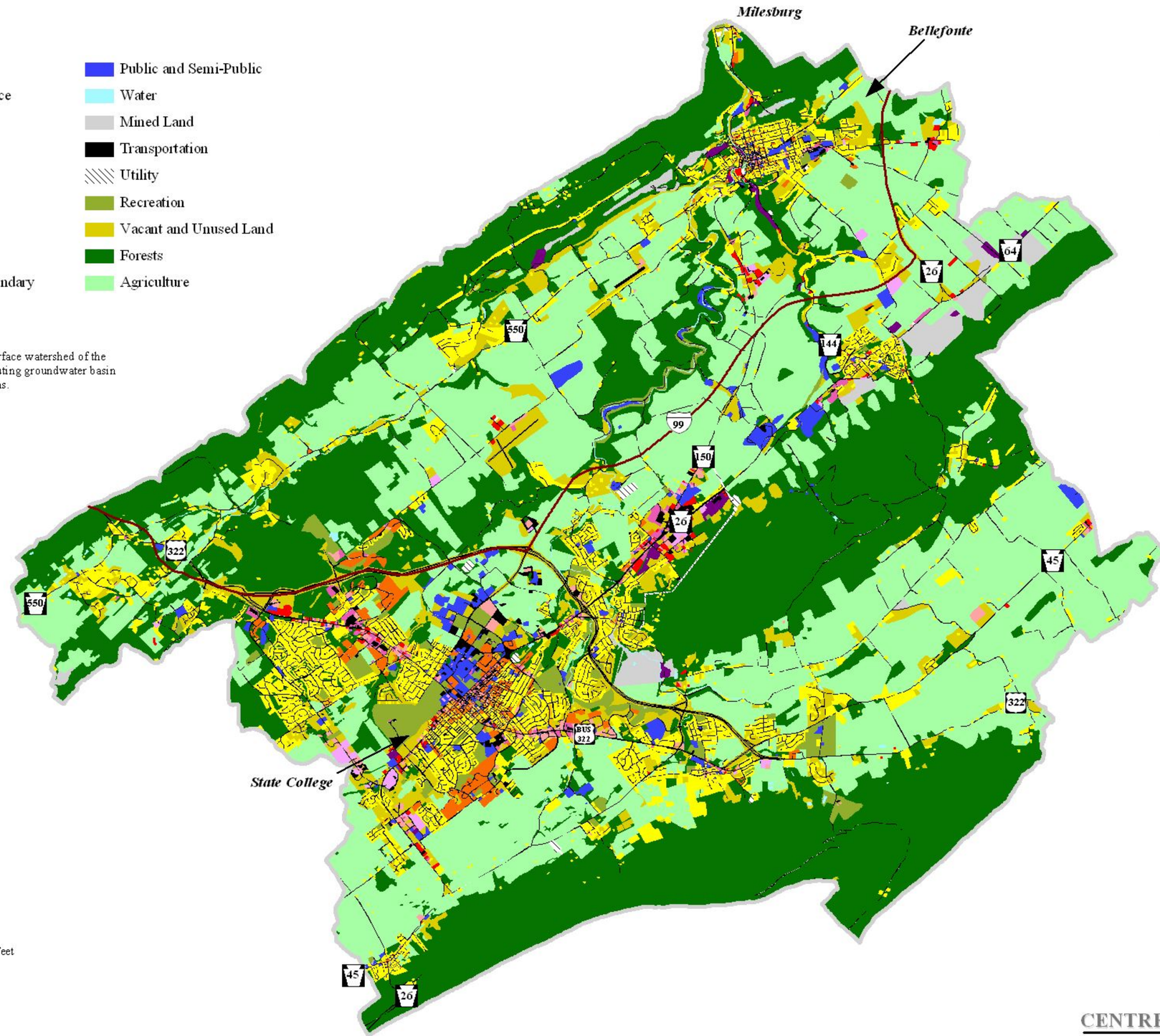
Downstream communities may feel that removing either the 50- or 100-year design storms from stormwater detention requirements may produce an adverse affect. However, the data do not generally support such a statement. The studies conducted herein have indeed validated the general increase of runoff peak rates due to development (refer to the recharge analysis section) within the basin. However, these are increases in non-extreme events. Analysis of past stormwater management practices indicates that, from a stormwater management perspective, we need to focus on these non-extreme events to eliminate more frequent nuisance flooding problems. Because land use has a very limited influence on extreme runoff events such as the 100-year event, land development activities do not significantly increase the magnitude of these less frequent flood events. It is interesting to note that three (3) of the four (4) largest runoff events, which occurred in this century (1912, 1936, 1972, and 1996) have occurred due to snowmelt runoff events. Under these conditions, most land areas within the watershed responded like impervious areas. Additionally, the largest storm of record, the flood of 1936, occurred prior to any major development in the basin, and was most likely what could

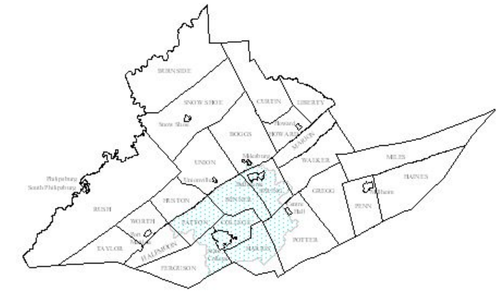
be considered a 100-year return period flood event. Putting a better level of protection on the more frequent events will be much more beneficial for all downstream residents.

SPRING CREEK WATERSHED STORMWATER MANAGEMENT PLAN

- Single Family Residence
- Two to Four Family Residence
- Multi-Family Residence
- Retail
- Services
- Heavy Commercial
- Heavy Industry
- Light Industry
- Spring Creek Watershed Boundary
- Public and Semi-Public
- Water
- Mined Land
- Transportation
- Utility
- Recreation
- Vacant and Unused Land
- Forests
- Agriculture
- Future I-99 Route

* This map reflects land use within the surface watershed of the Spring Creek drainage basin. The contributing groundwater basin extends beyond this boundary in some areas.





Spring Creek Watershed
Centre County, PA

One of the greatest influences on stormwater volume and water quality within a watershed is land use. Land use in the Spring Creek watershed is quite varied as agriculture and forested lands dominate a portion of the drainage basin while urban and suburban landscapes are equally visible in other areas.

As land is converted from less to more intense uses, agriculture to commercial for example, the resulting rooftops, roads, driveways, parking lots and sidewalks can disrupt natural drainage systems by preventing precipitation from soaking into the soil. The end product is stormwater runoff that travels over the land surface causing flooding, erosion, and the transport of pollutants into our high quality waterways. Groundwater recharge can also be negatively impacted.

The goal of this planning initiative is to establish a means for effectively managing stormwater quantity and quality in the Spring Creek Watershed by balancing the reality of landuse changes with the need to protect valuable streams and groundwater supplies.

0 3000 6000 9000 12000 Feet

Scale: 1 inch = 9000 feet

Projection: PA State Plane (North)
Datum: NAD83

SOURCES: Centre County Planning Office (1995)
January 2000

Prepared by the Centre County Planning Office.
This plan, in part, is funded by a grant from the Pennsylvania
Department of Environmental Protection, Bureau of Stormwater Management.

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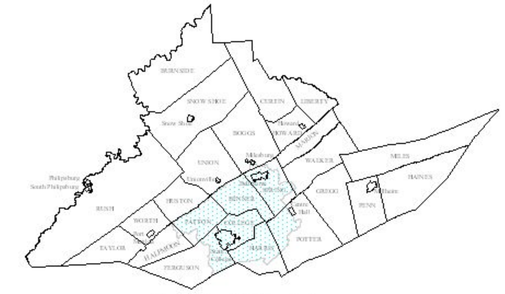
EXHIBIT 2-1: Existing Land Use Map

SPRING CREEK WATERSHED STORMWATER MANAGEMENT PLAN

Spring Creek Watershed
Municipal Boundaries
Future I-99 Route
Streams
Wetlands
Floodplain

NOTE: Floodplain and wetland boundaries shown here are approximate. Actual regulated floodplain limits are delineated on FEMA flood insurance maps available in each municipality. Wetland areas are more accurately delineated on the National Wetland Inventory Maps prepared by the U.S. Department of Interior, Fish and Wildlife Service.



Spring Creek Watershed
Centre County, PA

0 3000 6000 9000 12000 Feet
Scale: 1 inch = 9000 feet

Projection: PA State Plane (North)
Datum: NAD83

SOURCES: Federal Emergency Management Agency, National Wetland Inventory and PennDOT
January 2000

Prepared by the Centre County Planning Office.
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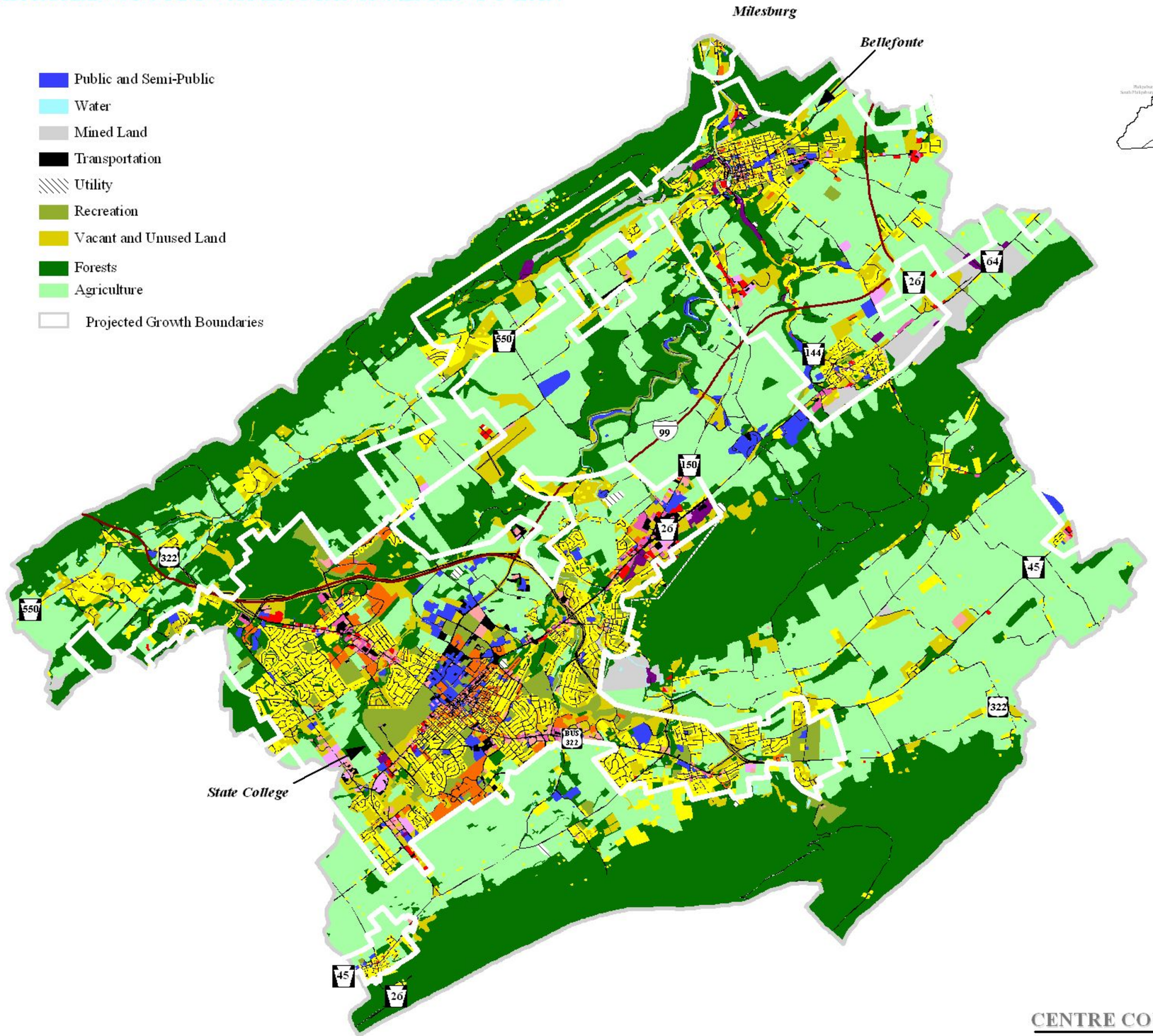
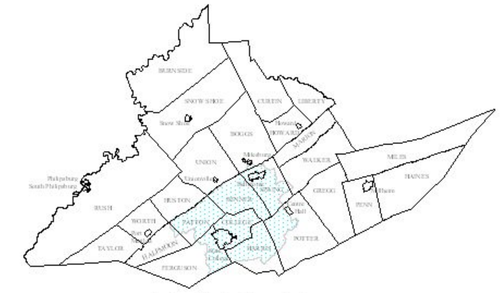
EXHIBIT 2-10: Floodplain and Wetland Areas Map

SPRING CREEK WATERSHED STORMWATER MANAGEMENT PLAN



- Single Family Residence
- Two to Four Family Residence
- Multi-Family Residence
- Retail
- Services
- Heavy Commercial
- Heavy Industry
- Light Industry
- Spring Creek Watershed Boundary
- Future I-99 Route
- Public and Semi-Public
- Water
- Mined Land
- Transportation
- Utility
- Recreation
- Vacant and Unused Land
- Forests
- Agriculture
- Projected Growth Boundaries



Spring Creek Watershed
Centre County, PA

0 3000 6000 9000 12000 Feet

Scale: 1 inch = 9000 feet

Projection: PA State Plane (North)
Datum: NAD83

SOURCES: Centre County Planning Office (1995)
January 2000

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EXHIBIT 2-2: Projected Growth Boundaries Map

SPRING CREEK WATERSHED STORMWATER MANAGEMENT PLAN

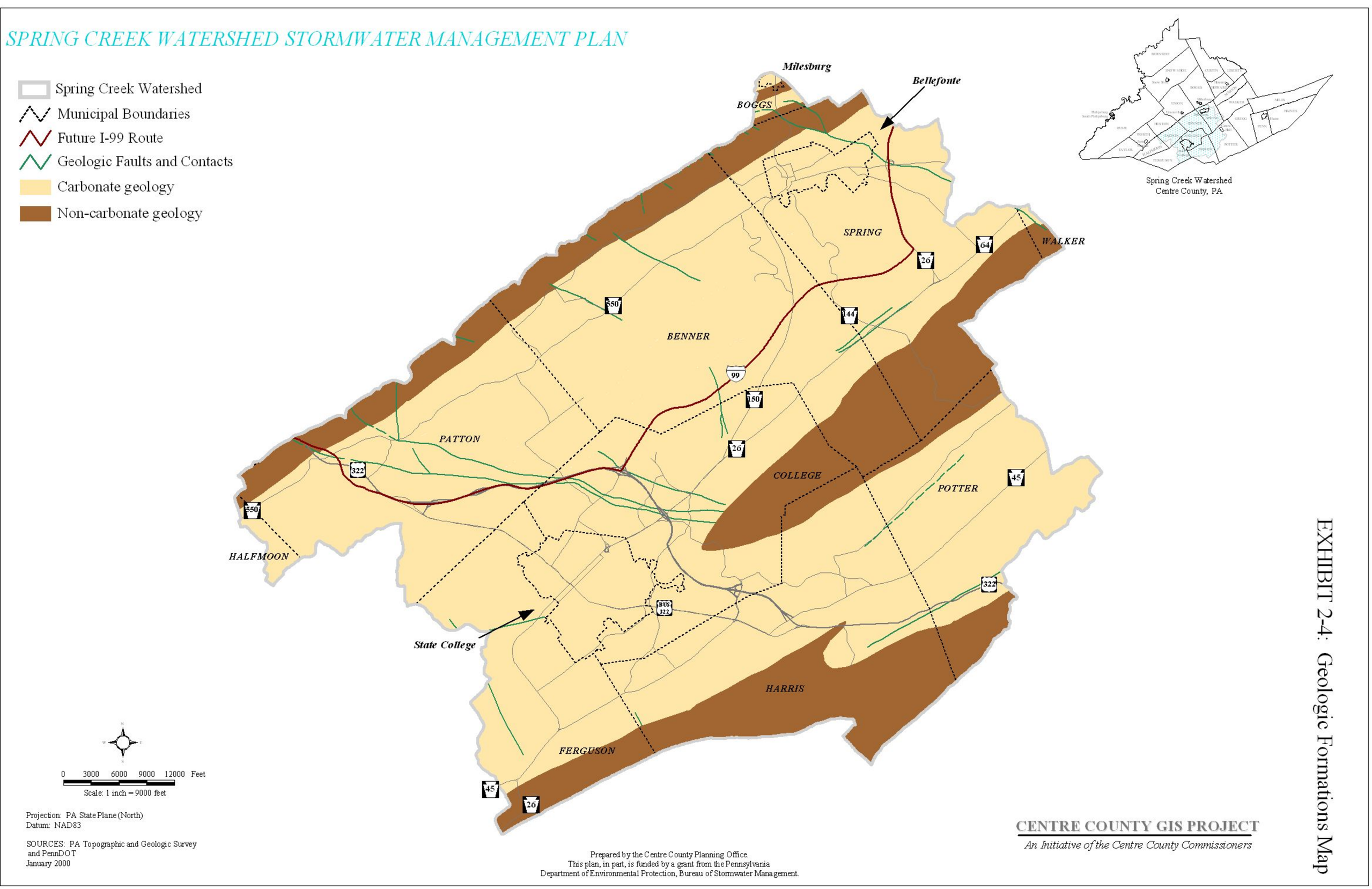
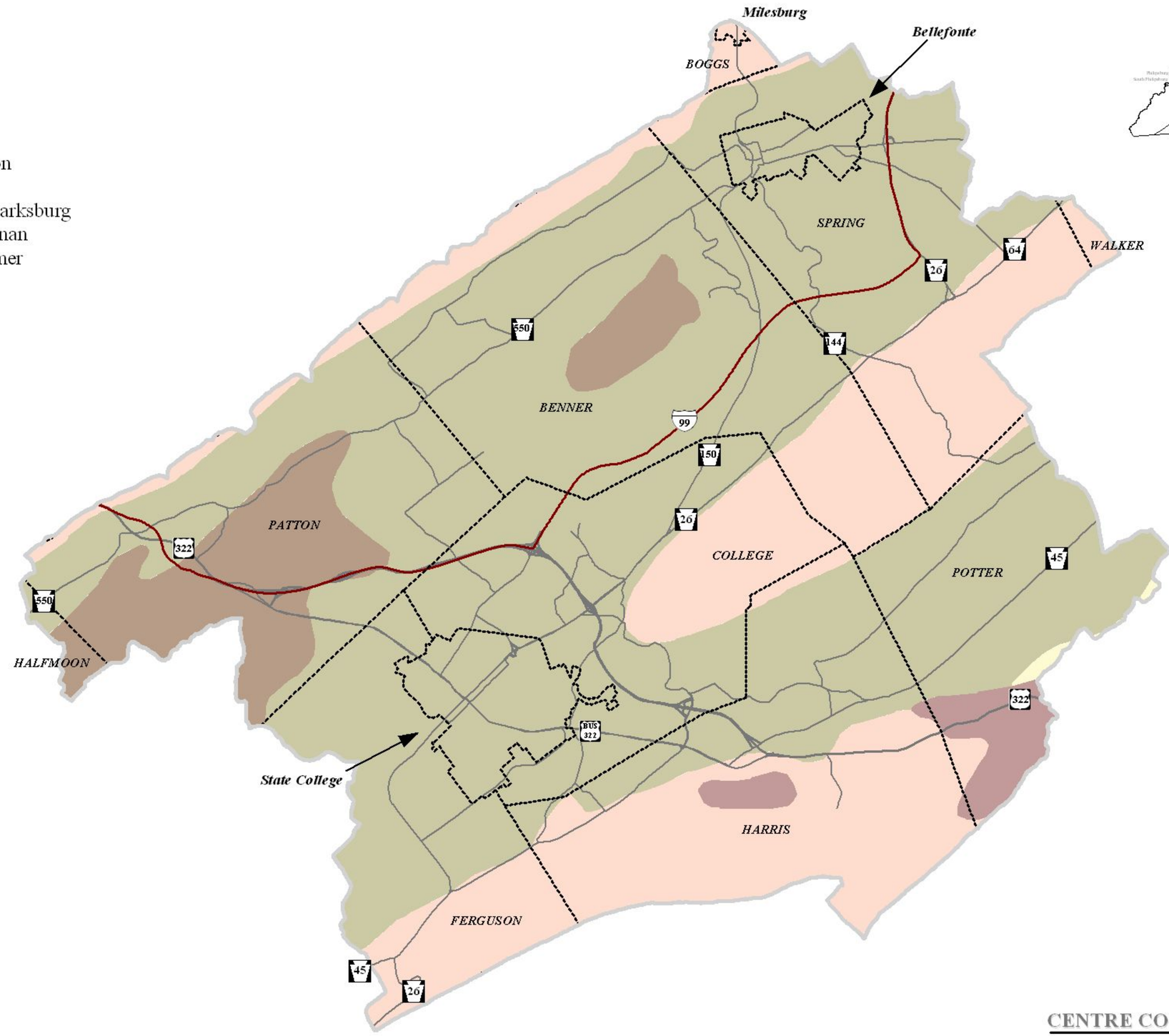


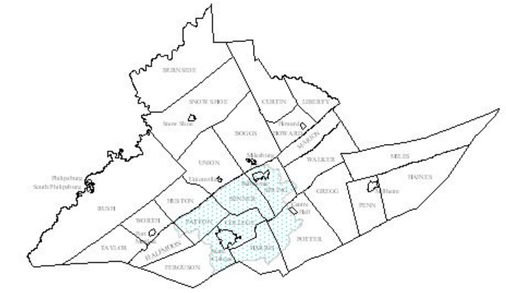
EXHIBIT 2-4: Geologic Formations Map

SPRING CREEK WATERSHED STORMWATER MANAGEMENT PLAN

Spring Creek Watershed
Municipal Boundaries
Future I-99 Route

Soil Associations
- Berks-Weikert-Bedington
- Edom-Millheim-Calvin
- Hagerstown-Duffield-Clarksburg
- Hazleton-Dekalb-Buchanan
- Morrison-Hazleton-Clymer





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Scale: 1 inch = 9000 feet

Projection: PA State Plane (North)
Datum: NAD83

SOURCES: STATSGO (U.S.D.A, Natural Resources Conservation Service) and PennDOT
January 2000

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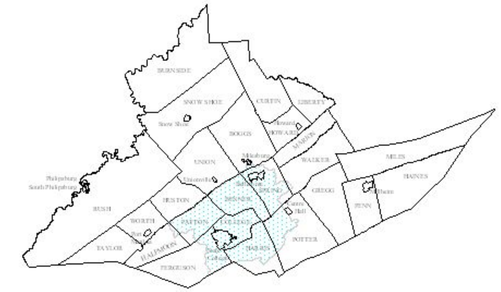
EXHIBIT 2-3: State Soil Geographic Database Map

SPRING CREEK WATERSHED STORMWATER MANAGEMENT PLAN

Spring Creek Watershed
Municipal Boundaries
Future I-99 Route

Karst Features
- cave
- sinkhole
- surface depression
- surface mine



Spring Creek Watershed
Centre County, PA

0 3000 6000 9000 12000 Feet
Scale: 1 inch = 9000 feet

Projection: PA State Plane (North)
Datum: NAD83

SOURCES: PA Topographic and Geologic Survey
and PennDOT
January 2000

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EXHIBIT 2-5: Karst Features Map

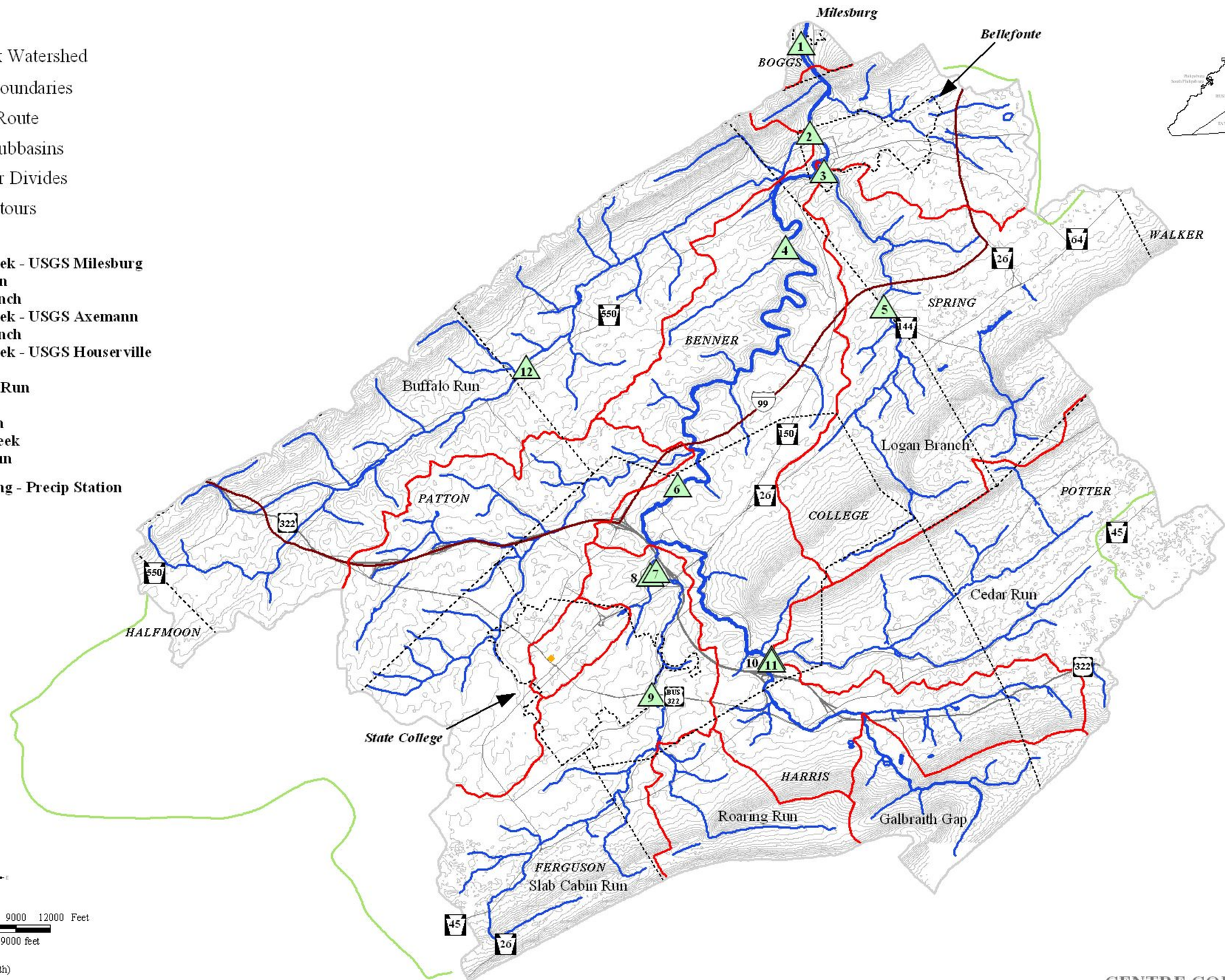
SPRING CREEK WATERSHED STORMWATER MANAGEMENT PLAN

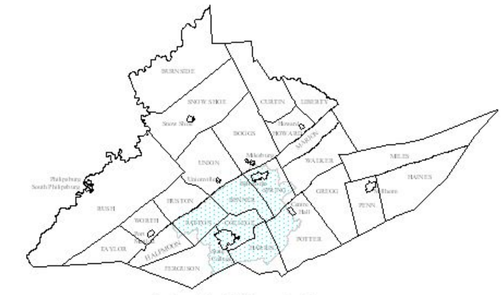
Spring Creek Watershed
Municipal Boundaries
Future I-99 Route
Watershed subbasins
Groundwater Divides
50 Foot Contours

Stream Gages

1 = Spring Creek - USGS Milesburg
2 = Buffalo Run
3 = Logan Branch
4 = Spring Creek - USGS Axemann
5 = Logan Branch
6 = Spring Creek - USGS Houserville
7 = Slab Cabin
8 = Thompson Run
9 = Slab Cabin
10 = Cedar Run
11 = Spring Creek
12 = Buffalo Run

Walker Building - Precip Station





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Scale: 1 inch = 9000 feet

Projection: PA State Plane (North)
Datum: NAD83

SOURCES: U.S. Geologic Survey (USGS), PennDOT, and Clearwater Conservatory
January 2000

Prepared by the Centre County Planning Office.
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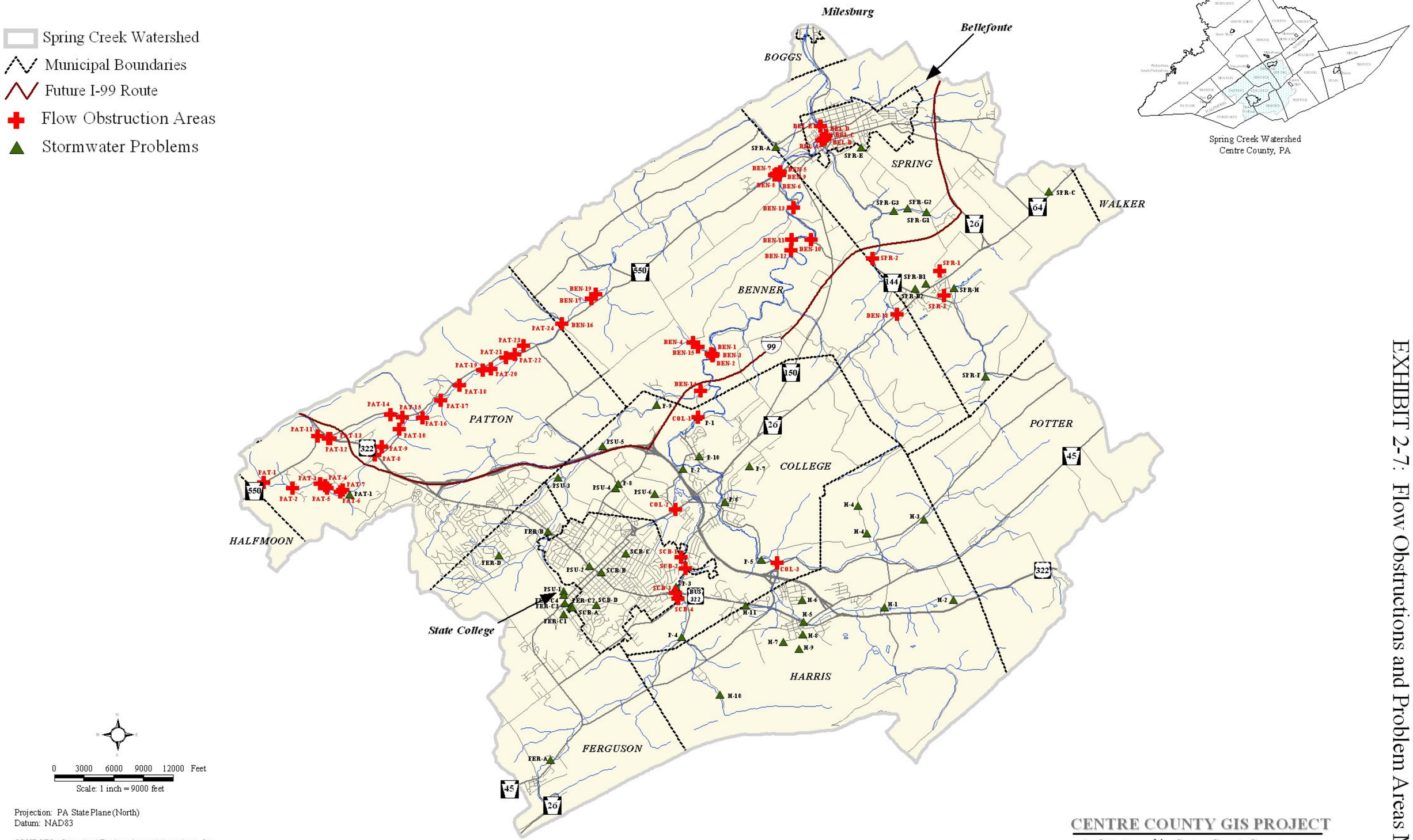
EXHIBIT 2-6: Hydrologic Subwatershed Boundaries Map

SPRING CREEK WATERSHED STORMWATER MANAGEMENT PLAN



Projection: PA State Plane (North)
Datum: NAD83

SOURCES: Sweetland Engineering and Associates, Inc. and PennDOT
January 2000

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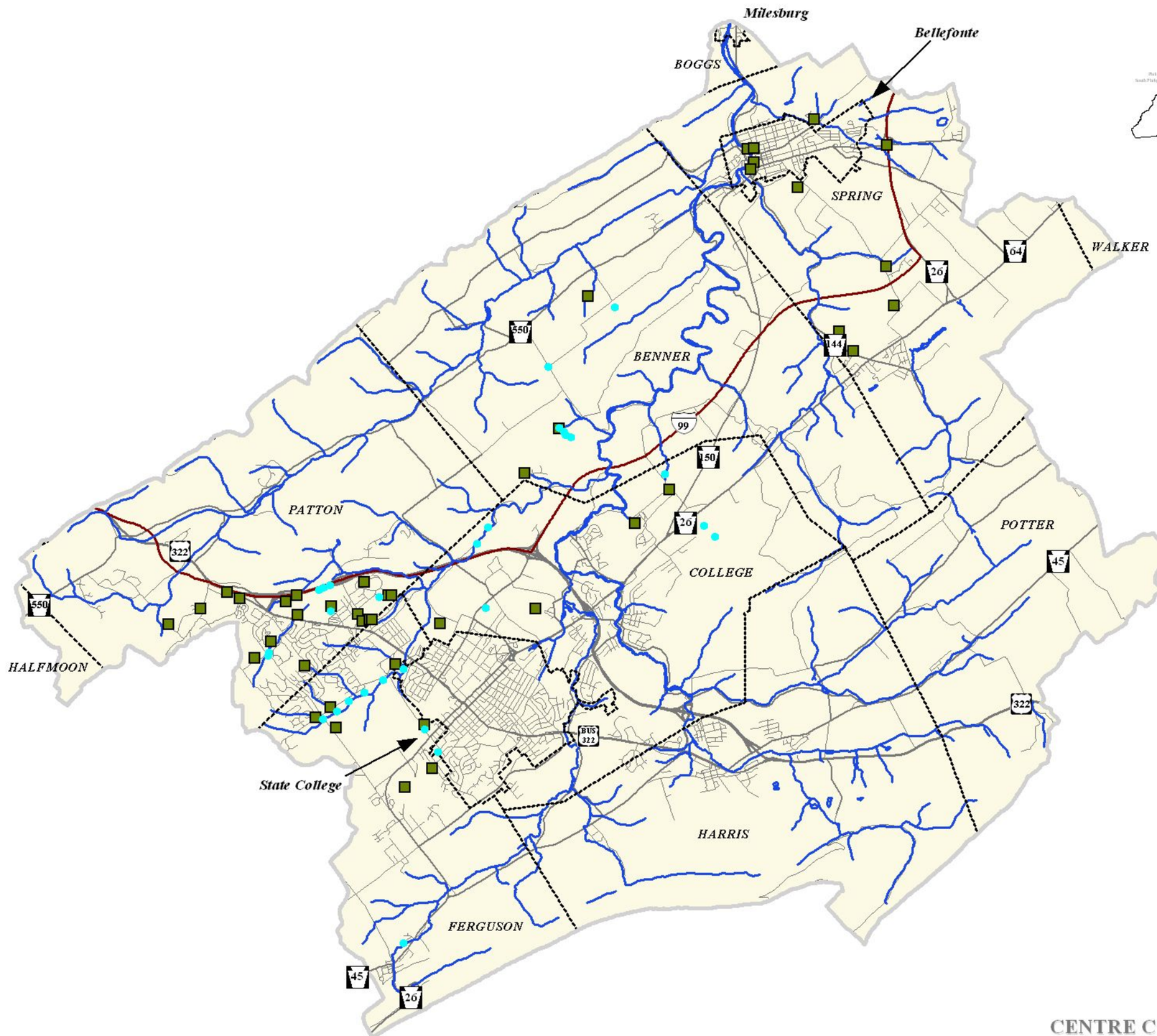
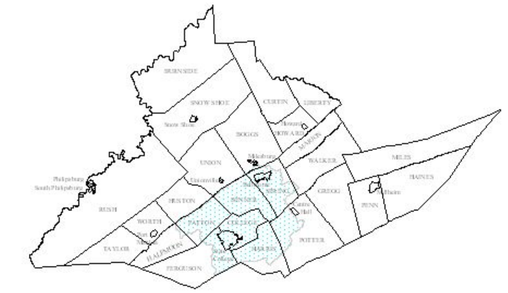
EXHIBIT 2-7: Flow Obstructions and Problem Areas Map

SPRING CREEK WATERSHED STORMWATER MANAGEMENT PLAN

Spring Creek Watershed
Municipal Boundaries
Future I-99 Route
Major Detention Facilties
Karst Problem Areas





Spring Creek Watershed
Centre County, PA

0 3000 6000 9000 12000 Feet
Scale: 1 inch = 9000 feet

Projection: PA State Plane (North)
Datum: NAD83

SOURCES: Sweetland Engineering and Associates, Inc. and PennDOT
January 2000

Prepared by the Centre County Planning Office.
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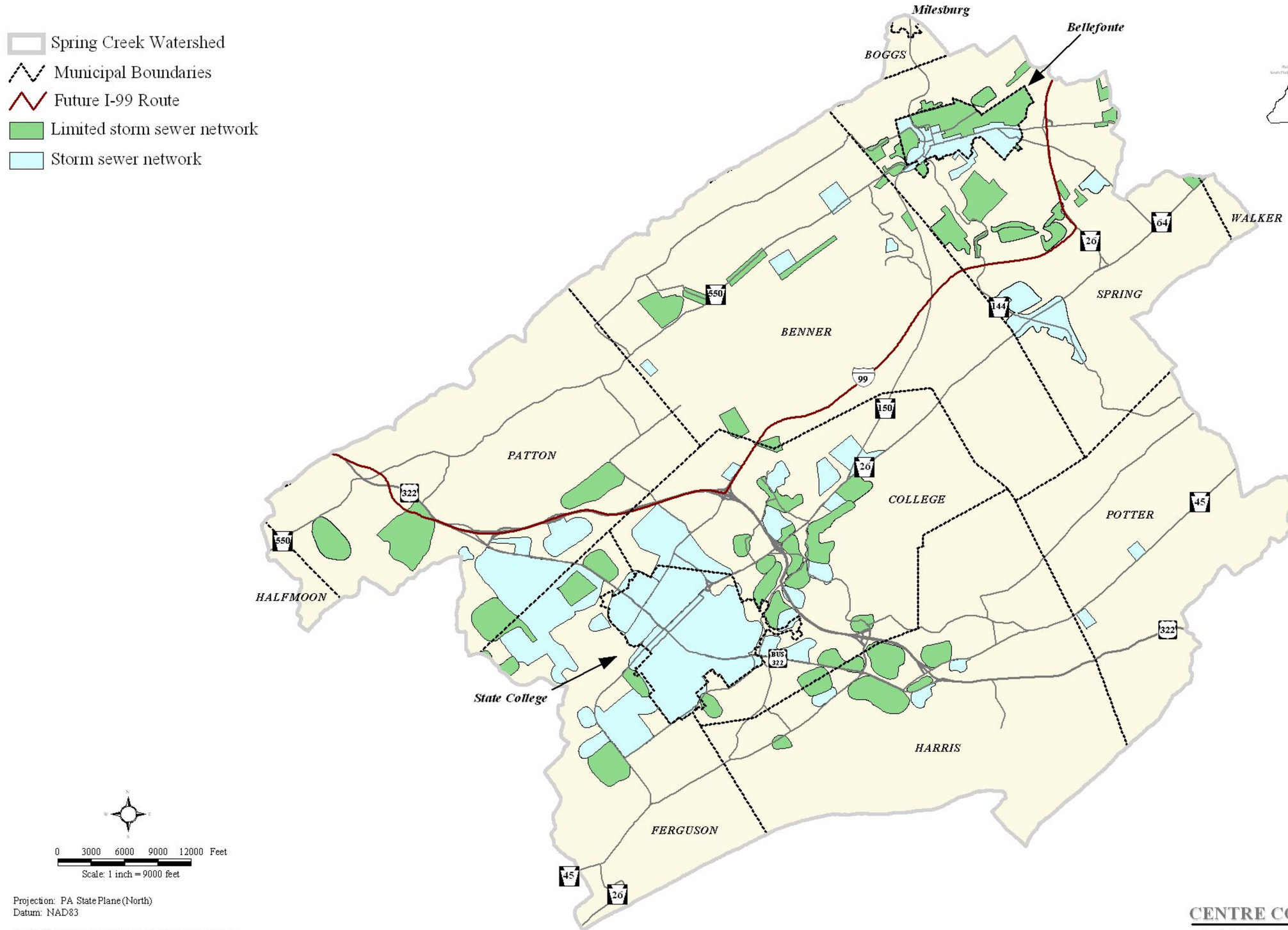
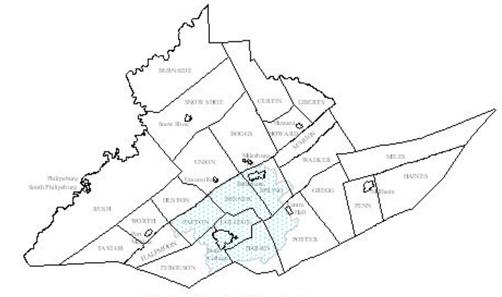
EXHIBIT 2-8: Major Detention Facilities & Karst Problem Areas Map

SPRING CREEK WATERSHED STORMWATER MANAGEMENT PLAN





Projection: PA State Plane (North)
Datum: NAD83

SOURCES: Sweetland Engineering and Associates, Inc. and PennDOT
January 2000

Prepared by the Centre County Planning Office.
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EXHIBIT 2-9: Storm Sewer Networks Map