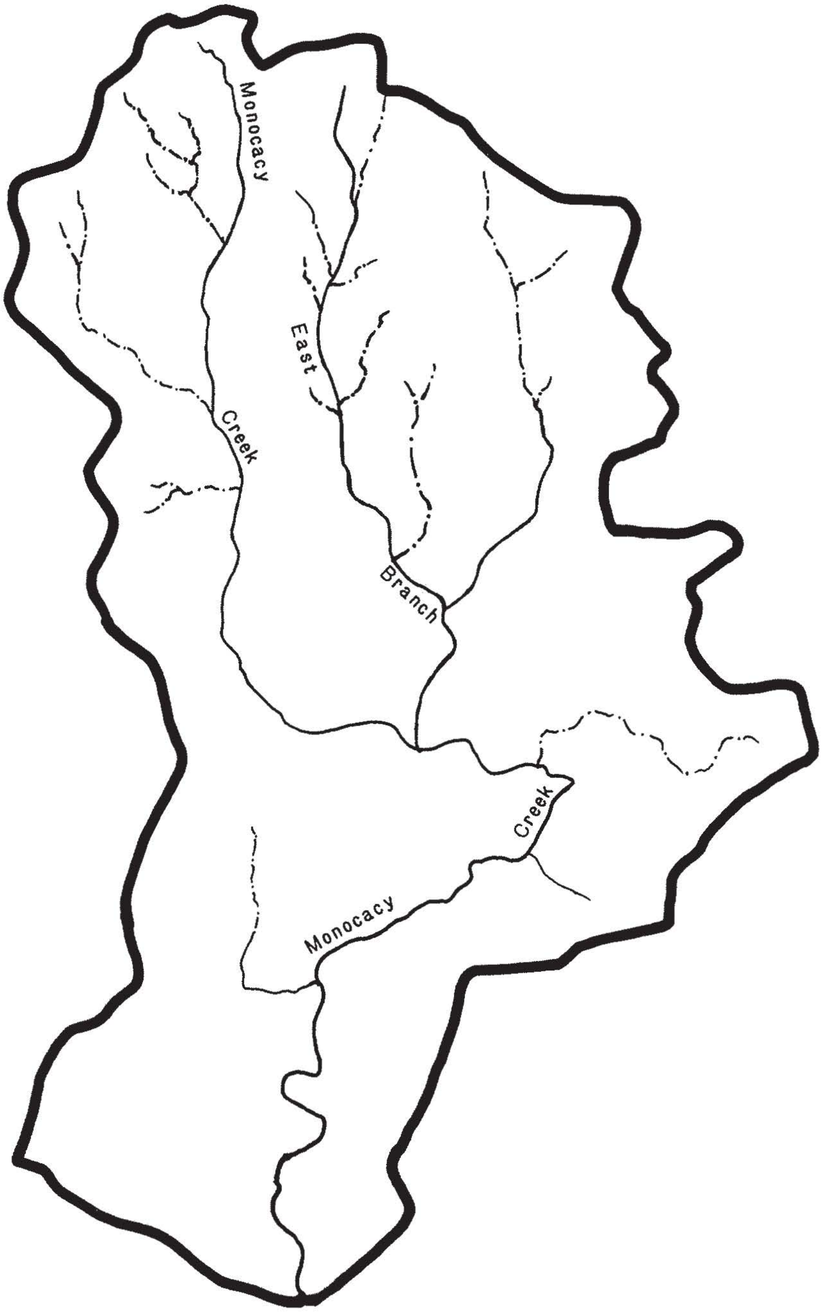
MONOCACY CREEK WATERSHED

- ACT 167 -

STORM WATER MANAGEMENT PLAN



March 1989

JOINT PLANNING COMMISSION LEHIGH–NORTHAMPTON COUNTIES

MONOCACY CREEK WATERSHED

- ACT 167 -

STORM WATER MANAGEMENT PLAN

This Plan has been prepared by the Joint Planning Commission Lehigh-Northampton Counties on behalf of Northampton County and Lehigh County. It contains revisions based upon the review comments received, if any, from the Monocacy Creek Watershed Advisory Committee, affected municipalities, general public, Northampton County Council and Lehigh County Board of Commissioners.

The preparation of this report was financed through the Pennsylvania Storm Water Management Grant Assistance Program administered by the Department of Environmental Resources. This grant program provides for seventy-five percent (75%) State funding with the twenty-five percent (25%) local match provided by Northampton and Lehigh Counties.

Final Plan March 1989
Adopted by Northampton County August 1989
Adopted by Lehigh County September 1989
Approved by DER November 1989

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**Project Planner
*Staff for this report

ENGINEERING CONSULTANTS

David F. Kibler, Ph.D., P.E.
Gert Aron, Ph.D., P.E.

Pennsylvania State University

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CHAPTER I. INTRODUCTION

The Pennsylvania Storm Water Management Act, Act 167 of 1978, provides the framework for improved management of the storm runoff impacts associated with the development of land. The purposes of the Act are to encourage the sound planning and management of storm runoff, to coordinate the storm water management efforts within each watershed and to encourage the local administration and management of a coordinated storm water program. Key words in the above are "coordinate" and "watershed". To date, storm water management decisions have been made at the municipal level through enforcement of local ordinances based upon whatever storm runoff control philosophy the municipality opted to use. This type of approach might work very well if runoff conformed to municipal boundaries. However, runoff conforms only to watershed boundaries. For the Monocacy Creek, this means that ten Northampton County municipalities and one Lehigh County municipality have a stake in how storm water runoff is managed in the watershed. Without an effort to coordinate their efforts through a watershed analysis, the eleven municipalities establish a fragmented system of storm water management with uncertain results. At best, the fragmented system results in an inefficient process of runoff management whereby conservative engineering design would dictate "over control" of runoff. At worst, the fragmented approach could result in the creation of additional storm drainage problems and associated costs and hazards. These detrimental impacts could occur even though all eleven* municipalities are diligently administering and enforcing their ordinances. This is because the existing ordinances do not all require an analysis of impacts beyond municipal boundaries and the watershed-wide data base has not been available to quantify downstream impacts. Typically, therefore, storm water management decisions would be based simply upon "at-site" considerations which may or may not produce results consistent with proper storm water management on the watershed level.

The difference between at-site runoff control philosophy and the Act 167 watershed-level philosophy is the consideration of downstream impacts. Whereas the objective of typical at-site design would only be to control post-development peak runoff rates to predevelopment levels from the site itself, a watershed-level design would be geared towards maintaining existing peak flow rates in the entire drainage system. The latter requires knowledge of how the site relates to the entire watershed in terms of the timing of peak flows, contribution to peak flows at various downstream locations and the impact of the additional runoff volume generated by development of the site. The proposed watershed-level runoff control philosophy is based on the assumption that runoff volumes will increase with development.

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*The Borough of Chapman does not have its own subdivision ordinance and the JPC administers the Northampton County ordinance within the Borough.

Rather than specifically attempt to reduce post-development volume, the Plan is intended to "manage" the increase in volumes such that peak rates of runoff throughout the watershed are not increased.

Act 167 storm water management on a watershed level will provide a significant step forward in the sound management of the runoff impacts of new development. The storm runoff control strategy established by an Act 167 plan provides for new development to occur while ensuring that existing drainage problems are not aggravated nor new problems created. It will not, however, eliminate storm drainage problems or flooding. To effectively implement an Act 167 program it is necessary to understand the following limitations of the process as well as the strengths:

- An Act 167 plan is not an engineering design document but provides an engineering framework for individual site evaluation and design.
- Storm runoff criteria are based on controlling "design" storm events applied uniformly over the entire watershed. Natural storms, which may vary in duration, intensity and total depth of rainfall throughout the watershed, may, in certain instances, create runoff events which cannot be effectively controlled.
- The runoff control criteria developed as part of an Act 167 plan will not correct existing drainage problem areas.
- An Act 167 plan will not prevent the inundation of floodplain areas. These areas are intended by nature to carry storm runoff.

It is also important to understand that an Act 167 plan is not a land use plan. Runoff controls developed in the Plan are not based upon controlling the location, type, density or rate of development throughout the watershed. The storm water runoff peformance standards are based on the assumption that development will occur throughout the watershed. The plan is designed to provide for new development yet control the associated storm runoff impacts.

The most important aspect of an Act 167 plan is that it establishes a process for decision-making. It establishes the existing interrelationships between the various parts of a watershed in terms of peak flows and the "timing" of those peak flows. The peak flows and timing relationships provide for development of a runoff control philosophy geared towards minimizing the storm runoff impacts of new development.

Act 167 is essentially a three-step process of runoff control which works as follows:

1 - Documentation of the existing state of storm runoff in the watershed. Included herein is the documentation of the existing physical characteristics of the watershed (e.g. land use, soils, slopes, storm sewers, etc.), documentation of existing storm drainage problems and flow obstructions and documentation of the peak flow and timing relationships. The existing condition establishes the baseline situation against which all runoff control measures will be judged.

2 - Preparation of the Plan to control storm runoff from new development. The Plan includes runoff control performance standards for new development <u>and</u> a process for site specific evaluation and design. The performance standards do not dictate the control methods to be used but rather will indicate the necessary end product. The runoff control philosophy is designed to prevent new problem areas from developing. Stated otherwise, the runoff control philosophy seeks to ensure that peak runoff rates throughout the watershed will not increase with development. Successful implementation of the control philosophy would mean continuation of the status quo runoff situation (i.e. "freeze the action").

3 - Development of priorities for implementation. With the accomplishment of the first two aspects of the Act 167 process the third aspect involves developing a prioritized list of actions aimed at improving the current state of storm runoff in the watershed. Essentially, this means preparing a strategy for dealing with the existing storm drainage problem areas within each municipality.

One especially important aspect of the Act 167 process is the need to periodically update the plan. Act 167 specifies that a plan must be updated every five years, at minimum. This guarantees a dynamic system of watershed runoff control sensitive to changing watershed characteristics.

The "Monocacy Creek Watershed - Act 167 - Storm Water Management Plan" has been prepared for Northampton County by the Joint Planning Commission. Northampton County has designated the JPC to prepare the watershed plans for all County watersheds on the County's behalf. The technical aspects of plan preparation require the use of consulting engineering assistance. Two Pennsylvania State University Professors, Dr. David F. Kibler and Dr. Gert Aron, have been selected to provide the necessary technical assistance. The engineering support for the project has been provided on an "over-the-shoulder" basis whereby JPC staff is responsible for performing the work consistent with an approved methodology and subject to consultant review. In this way, the technical integrity of the plan is maintained while minimizing project cost.

To ensure the involvement of the municipalities and agencies which will be impacted by the Storm Water Management Plan, Act 167 requires that a Watershed Plan Advisory Committee be formed. The purposes of the Committee are to assist in the development of the Plan and familiarize the municipalities involved with the storm water management concepts evolving from the plan process. Each municipality in the watershed plus the County Conservation District are required to be represented on the Committee. Representation by additional agencies and interest groups are optional at the discretion of the county. Listed below are the names of the persons and their affiliations who participated on the Monocacy Creek Watershed Plan Advisory Committee:

Municipality/Organization	Name
Borough of Bath	Ira Faro
City of Bethlehem	Gary Falasca
Bethlehem Township	Carl DiCello
Bushkill Township	Phillip Parsons
Borough of Chapman	Curtis Fehnel
East Allen Township	Isabelle French
Hanover Township (L)	Sandra Kutos
Hanover Township (N)	Gertrude Fox
Lower Nazareth Township	David Heinly
Moore Township	Mike Schmalzer
Upper Nazareth Township	No representative
Northampton County Conservation District	Robert Jones
Trout Unlimited	Vince Guida
Wildlands Conservancy	Thomas Kerr
USDA Soil Conservation Service	Barry Frantz
Monocacy Creek Watershed Association	John Iverson
PA Fish Commission	Terry Hannold
Lehigh Valley Building Industry Association	Don Wohlsen

The general framework for the Monocacy Creek Act 167 Plan has been developed from three sources, namely Act 167 itself, the DER Storm Water Management Guidelines, which represent the Department's interpretation of the Act, and the several pilot watershed studies performed prior to the initiation of the State's regular program. In addition, the basic methodology used to quantify the watershed rainfall-runoff response function and to develop the runoff control criteria for new development has been adapted to the Monocacy Creek from selected pilot studies - most notably the Allegheny County and Delaware County efforts - and from the Little Lehigh Creek Watershed Plan prepared for Lehigh County.

CHAPTER II: WATERSHED CHARACTERISTICS AND HYDROLOGIC RESPONSE

A. General Characteristics

The Monocacy Creek is a tributary of the Lehigh River located predominantly within Northampton County with a small area located in Hanover Township, Lehigh County. A location map of the Monocacy Creek Watershed is presented in Figure 1. The creek has a drainage area of 49.3 square miles. The watershed is comprised of the mainstem Monocacy Creek and the East Branch Monocacy Creek which are shown in Figure 2.

The headwaters of the creek are underlain by the Martinsburg shale formation, which consists of three distinct parts. Uppermost in the watershed the geology is primarily banded clay slate or shale with only small amounts of sandstone. Beds of sandstone dominate the middle Martinsburg area although some slaty beds are also present. The lowermost shale region contains the banded clay slate similar to the upper region, but with more sand and thinner beds. Below the Martinsburg shale formation in the watershed lies a vast area of predominantly limestone. Jacksonburg, Beekmantown and Allentown limestone make up approximately ninety percent (90%) of the geology of the lower two-thirds of the Monocacy Creek watershed. The three limestone formations are part of the Great Valley Section of the Valley and Ridge Physiographic Province. Figure 2 depicts the boundary between the shale and limestone geology.

The topography of the slate areas of the watershed is characterized by low, flat-topped hills dissected by the creek producing steep-sided valleys. There are numerous quarries throughout the slate areas, some of which are partially filled with water creating small lakes. The topography of the limestone portion of the watershed is very flat with gently sloping valleys. Sinkholes and closed depressions occur frequently. During drier months, certain stretches of the creek will disappear to groundwater only to reappear from springs further downstream. Groundwater in the limestone region flows mainly in well-defined channels formed by solution of limestone along joints. Springs emerge throughout the lower portion of the limestone region, especially at contact between consolidated and porous or less compact material. An area of concentrated springs occurs as far upstream as Camel's Hump, a granitic outcrop near the northern boundary of the City of Bethlehem between State Routes 191 and 512. There are nine (9) documented springs in the vicinity of Camel's Hump and a total of sixty-four (64) springs located within the twelve (12) mile stretch of creek between Camel's Hump and the Lehigh River. The springs are known to substantially increase the Monocacy Creek's flow and their cooler temperature relative to surface flows in the warmer months helps to maintain the natural trout habitat.

FIGURE 1
MONOCACY CREEK LOCATION MAP

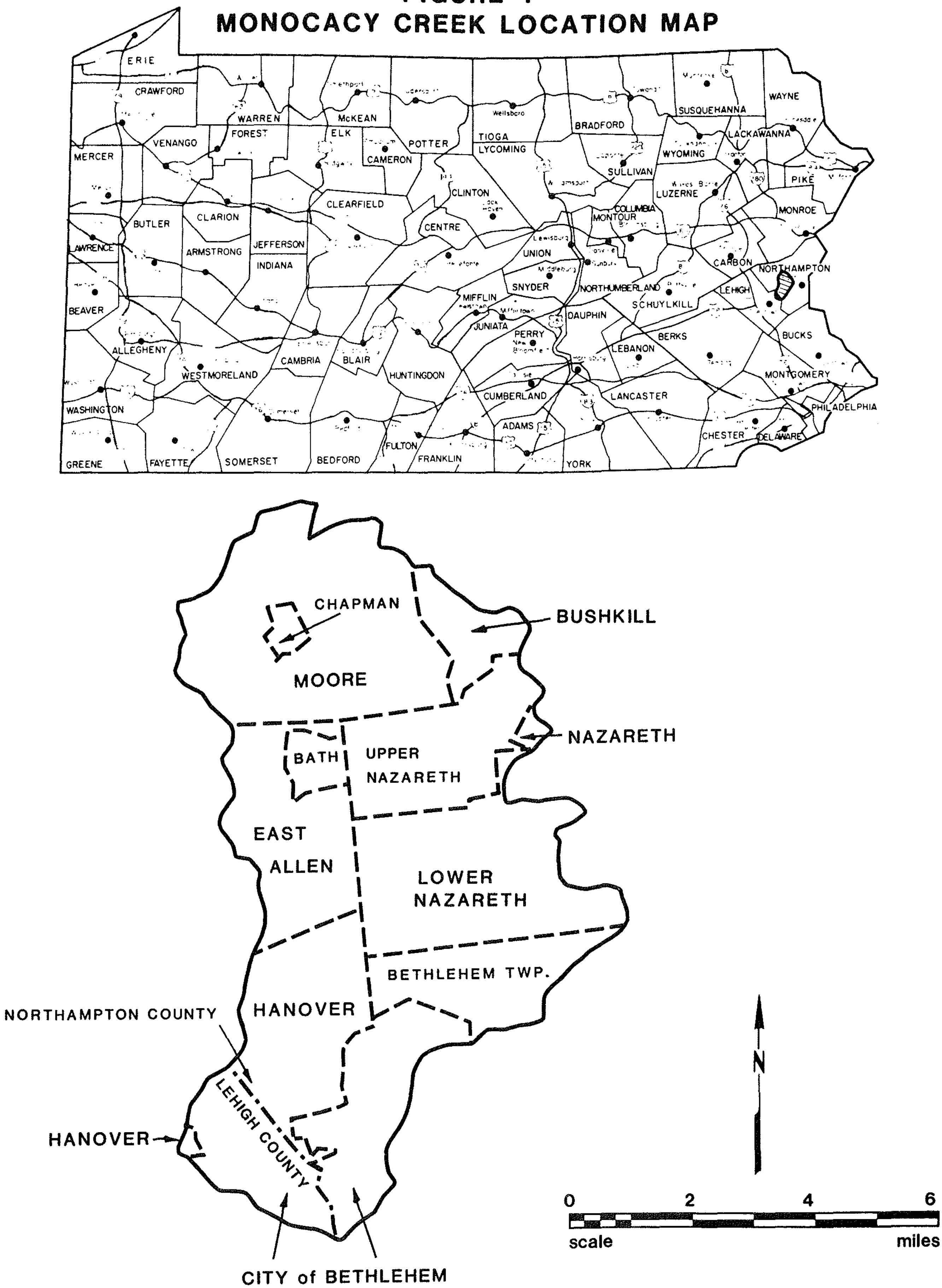
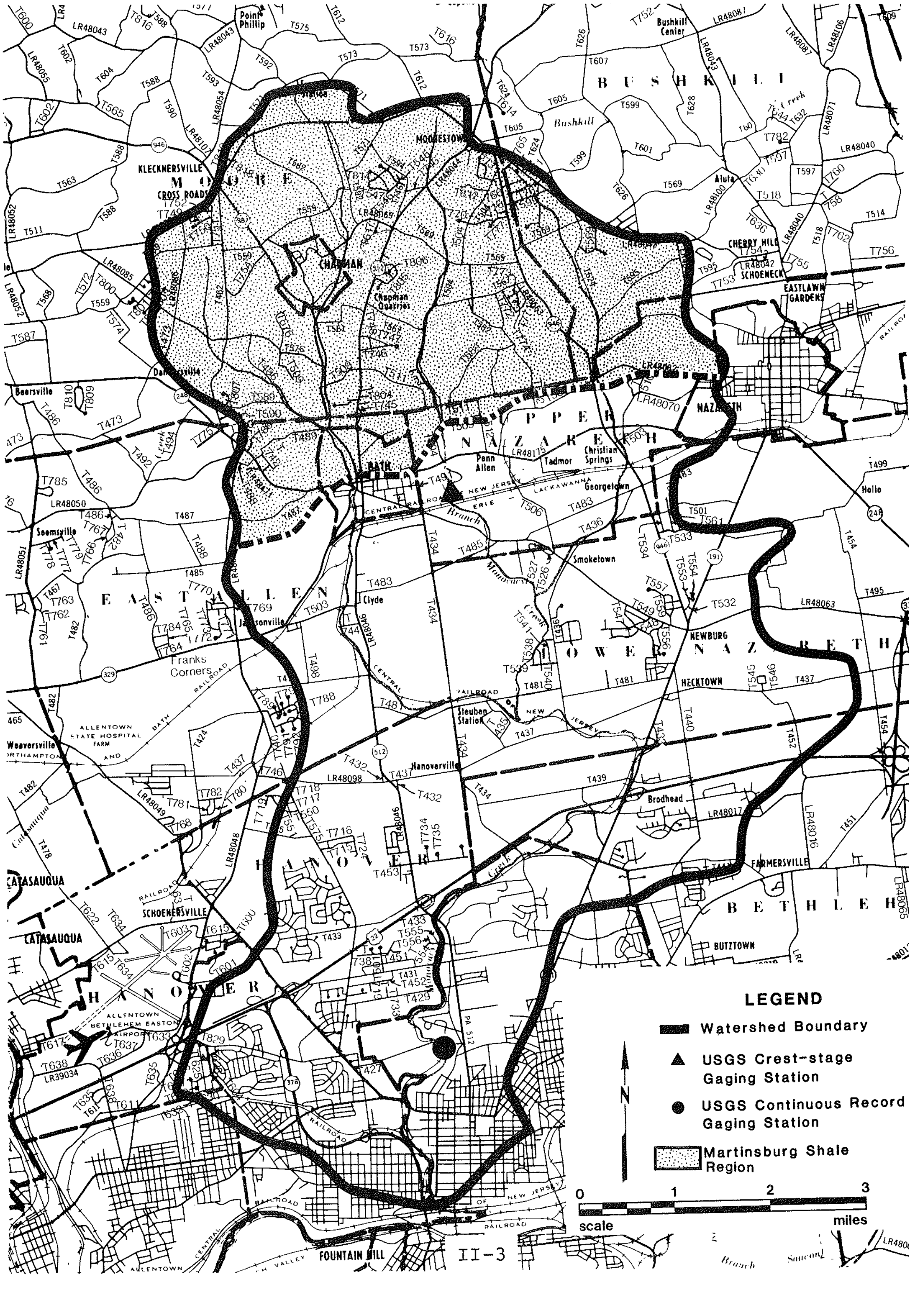


FIGURE 2
MONOCACY CREEK WATERSHED MAP



The basic reason for spring formation near Camel's Hump is the junction of the broad band of Beekmantown limestone to the north with Byram granitic gneiss underlying Camel's Hump and the region immediately to the west. The metamorphic gneiss restricts the passage of groundwater flowing from the limestone region causing it to exit in a series of springs in the immediate vicinity of the Monocacy Creek, which turns west, parallel to the topographic barrier at this point.

Average annual precipitation in the Monocacy Creek Watershed for the period of 1946-1962 was 45.7 inches, of which 15.6 inches (34.1%) became runoff. Evapotranspiration for the basin has been estimated at an annual average of 26.4 inches.

The Department of Environmental Resources (DER) has designated water quality criteria which are designed to protect the water uses within a given watershed. The Monocacy Creek has two water uses that are protected. One is the cold water fishes (CWF) category. This category helps to protect aquatic life in that it deals with the maintenance and/or propagation of fish species and flora and fauna which are native to cold water habitats. The other use deals with the special protection of high quality waters (HQW). High quality waters are considered as a stream or watershed with excellent quality water and environment features that require special protection.

The DER criteria state that high quality waters are to be protected and maintained at their existing quality or enhanced unless it can be shown that any increased discharge of any pollutant is justified as a result of economic or social development which is of significant public value. The best available treatment and land disposal technologies must be used where economically feasible and environmentally sound.

The Monocacy Creek is used primarily for aesthetic and recreational purposes. It is known for its excellent fishing. In fact, the Monocacy Creek has recently been designated as a Trophy Trout stream in the area between the dam at Illicks Mill Road upstream to Bella Vista Road. The stream had to meet three requirements set by the State Fish Commission in order to be designated as such. The requirements are that the permission of the adjacent land owners must be granted; the stream must be able to reproduce trout naturally; and it must be controlled environmentally. The stream will not be stocked with trout because stocking the stream causes pressure on the stream and the stream would no longer be able to support the trout naturally. Fisherman will be permitted to use a fly, or any artificial lures, but no worms or other live bait. This designation will be in effect at the beginning of trout season in 1988.

Land use within the basin varies from predominantly urban land uses at the lower portion of the watershed (City of Bethlehem, near the mouth of the Monocacy) to more suburban/rural land uses in the northern, upstream portion of the watershed (Lower Nazareth, Upper Nazareth, East Allen, Moore and Bushkill Townships). An exception to this is the Borough of Bath, located near the top of the watershed, which is predominantly urbanized although it is relatively small. In total, the Monocacy Creek Watershed is approximately 50% urban/suburban land uses and 50% rural/agricultural land uses.

B. Hydrologic Response

The United States Geological Survey (USGS) has maintained a continuous record gaging station on the Monocacy Creek Mainstem within Monocacy Park since 1949. The station is located 2.1 miles upstream from the mouth at the Lehigh River just downstream of Illicks Mill Road as shown in Figure 2. The drainage area monitored at the gage location is 44.5 square miles. The gaging station records the depth of water in the creek at one-hour intervals. U.S.G.S. has prepared a "rating curve" for the gage location which relates water depth to actual flow in cubic feet per second (cfs). Water depth information for storm events can therefore readily be translated into flow rate data. Further, statistical analysis of the gaging station data can establish the probability of occurrence of flows of a given size. For example, the statistical analysis can determine the peak flow value which would be expected to occur on the average once every two years. This peak flow value would be termed the 2-year return period peak flow. Using the standardized probability relationships, the peak flow versus return period data can be established for a wide range of return periods. One such probability relationship, the Log Pearson Type III, has been applied to the 38-year Monocacy Creek gage record (through September 1987) to determine the peak flow-return period correlation presented in Table 1.

TABLE 1

Monocacy Creek Mainstem U.S.G.S. Gaging Station Frequency Data*

Return Period	Peak Flow
2-Year	574 cfs **
5-Year	1,133 cfs
10-Year	1,656 cfs
25-Year	2,529 cfs
50-Year	3,361 cfs
100-Year	4,369 cfs

*Source: Frequency Analysis by Dr. David F. Kibler.
**Cubic feet per second.

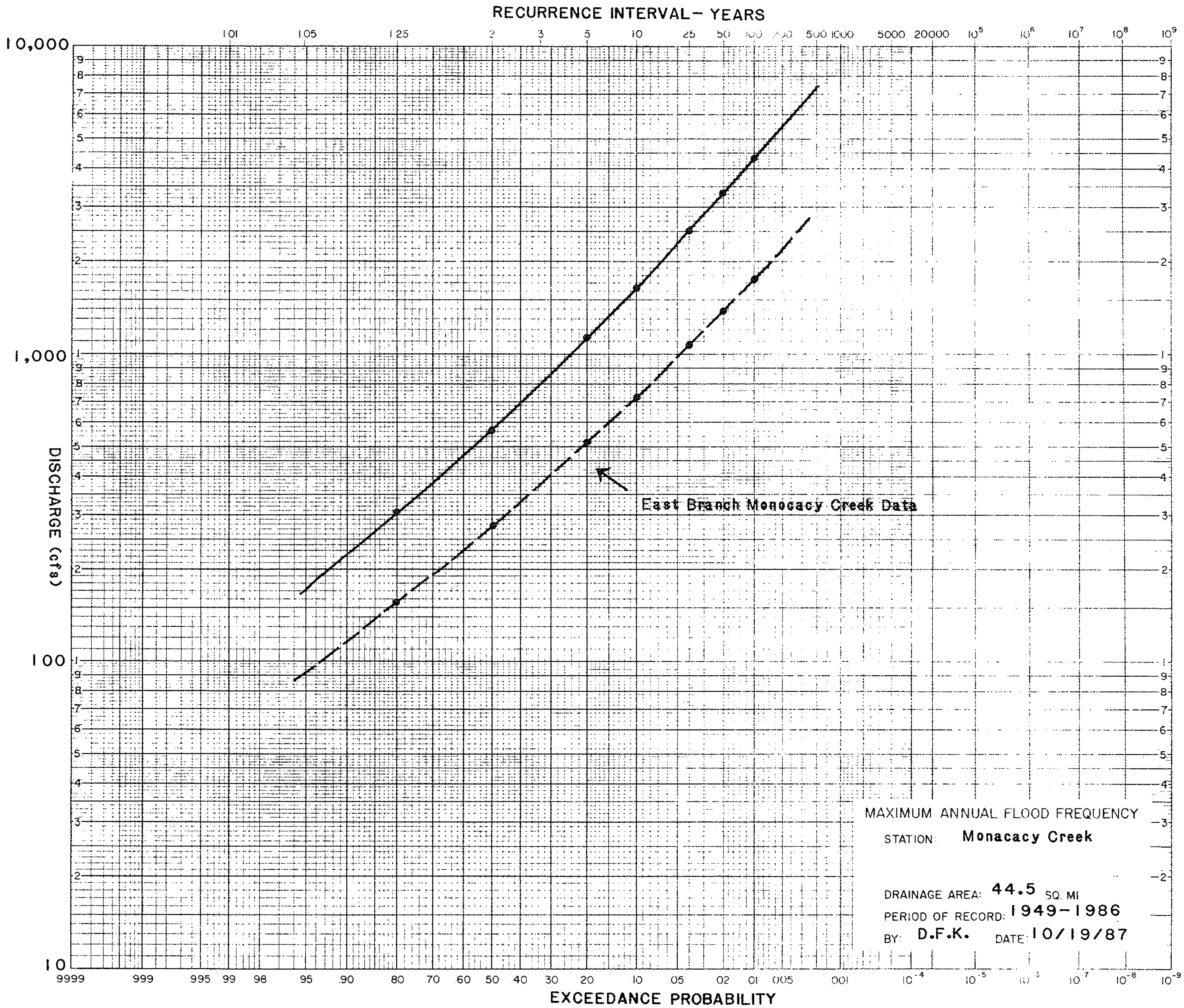
A grapical representation of the data or "frequency curve" is presented in Figure 3.

Note that with only 38 years of data, the ability to accurately define the 50- or 100-year return period runoff peak is questionable. An illustration of this is that the published 100-year return period peak flow for the Monocacy Creek gage is 2,907 cfs in the DER Water Resources Bulletin No. 13 (Oct. 1977) based upon 1949-1972 data. The largest flow of record occurred in January 1979 at a value of 3,490 cfs. With this peak flow (and the other annual peaks through 1987) included in the probability analysis, the calculated 100-year event increased by approximately 50% (2,907 to 4,369 cfs).

The U.S.G.S. maintains another gaging station on the Monocacy Creek located on the East Branch immediately south of Newburg Road. This gage, however, is not a continuous record gage and it only records the peak depth of a given storm event and not the entire hydrograph. Data from this gage is valuable for comparison to the peak flows recorded downstream in Monocacy Park. The drainage area monitored by the East Branch gage is 5.35 square miles, which is almost entirely underlain by slate geology, and the period of record is 1963 to the present. Statistical analysis of the gage data using the Log Pearson Type III methodology yielded the frequency curve for the East Branch gage as shown in Figure 3.

Historical runoff data serves an important function in the development of a runoff control plan. The hydrologic runoff model of the watershed prepared for the purpose of watershed analysis must provide an adequate representation of the recorded watershed runoff response to be technically valid. In reviewing the historical data for the Monocacy Park continuous record gaging station, a definite pattern was readily apparent. The largest streamflows of record occurred predominantly during the winter months. In fact, sixteen (16) of the highest twenty (20) recorded runoff events occurred between January 21st and March 14th throughout the period of record. This phenomenon is somewhat understandable because the ground could be frozen during the winter months and produce more runoff. A compensating factor, however, is that the most intensive rainfall events tend to occur in the warmer months as thunderstorms or perhaps associated with hurricanes. The Monocacy Creek data is not typical in that it is so heavily skewed to the winter events. Presented in Table 2 is a listing of the highest twenty recorded runoff events at the downstream gage. The four non-winter events are marked with asterisks by the date.

FIGURE 3
MONOCACY CREEK FLOOD FREQUENCY CURVE AT U.S.G.S. GAGING STATION*



*Source: Frequency analysis by Dr. David F. Kibler

TABLE 2

Monocacy Creek Mainstem U.S.G.S. Gaging Station
Historical Peak Flow Data

Rank of Peak	Peak Flow cfs	Date of Storm	Approximate Return Period, Years
1	3,490	January 25, 1979	70 yr.
2	2,340	February 28, 1958	18
3	2,320	January 26, 1978	18
4	2,180	February 26, 1979	15
5	2,150	January 26, 1976	15
6	1,500	February 13, 1971	7
7	1,409	September 8, 1987*	6
8	1,340	February 26, 1962	6
9	1,310	February 25, 1979	6
10	1,310	May 30, 1984*	6
11	1,290	February 8, 1965	6
12	1,170	March 6, 1963	5
13	1,160	January 21, 1979	5
14	1,150	January 22, 1958	5
15	1,010	February 25, 1977	4
16	923	September 27, 1985*	3.5
17	915	January 21, 1959	3.5
18	811	August 8, 1982*	3
19	776	February 21, 1986	3
20	755	March 14, 1978	3

*Indicates non-winter event.

Two additional interesting things to note from Table 2 are that the largest flow associated with any non-winter storm (September 8, 1987) was only a six (6) year return period event and that all four of the non-winter events listed are in the decade of the 1980s. Regarding the former, the September 27, 1985 peak flow was produced by a rainfall depth of 7.85 inches. This rainfall is larger than that generally accepted to be the 100-year rainfall event! Stated otherwise, a storm considered so severe that it may only occur once every 100 years produced a Monocacy Creek streamflow which you would expect to have once every three and one-half years on the average. Conversely, the wintertime storm which produced the most severe flood of record of 3,490 cfs had a total rainfall of 1.51 inches, or less than that which would be expected every two years. It is clear, therefore, that the warm weather Monocacy Creek Watershed can act like a 50-square mile sponge, but, when frozen, can produce alarmingly high amounts of runoff from relatively small rainfall events.

Regarding the latter point mentioned above, the non-winter events have begun to get more severe resulting in 1980's data creeping into the record of highest flows. The importance of this point is as follows: If the Monocacy Creek only produced significant floods during winter (i.e. frozen) conditions, the need to prepare a watershed plan would be minimal since frozen ground and impervious areas created by development would produce nearly the same runoff. Therefore, virtually no stormwater controls on new development would be necessary to preserve the existing watershed conditions (the goal of the plan). However, the fact that four summer events this decade have become among those most severe would appear to indicate that new development is having an adverse impact and that proper runoff controls on future development are important. The Monocacy Creek Watershed Plan will provide criteria for controlled release of stormwater from new developments (in most cases) geared toward maintaining existing (non-frozen condition) peak flows. These criteria should also provide a beneficial impact on frozen condition peak flows through the runoff storage facilities incorporated into most new developments to meet the Plan criteria.

The prerequisite to the development of a runoff control strategy is the preparation of a hydrologic model which accurately simulates the recorded runoff response of the watershed to given rainfall conditions. Streamflow events of significance have been documented above. Rainfall data associated with each event is available from records kept by the National Weather Service (NWS). The NWS maintains a recording raingage at the Allentown-Bethlehem-Easton Airport which provides hourly rainfall data and documents the distribution of rain throughout the storm in addition to the total rainfall depth. Rainfall data for various historical events can be input to the hydrologic model and the calculated runoff event can be compared to the actual runoff recorded at the stream gage. To the extent that the calculated runoff differs from the recorded runoff, certain model parameters can be adjusted to better simulate the recorded streamflow. The process of model adjustment to match historical data is called calibration. Successful calibration would produce a hydrologic model that adequately simulates recorded streamflow for a broad range of rainfall events.

Eight (8) historical events were selected for calibration purposes. Of the eight, six were non-winter events and two were winter events. The results of the calibration attempts were dramatic but consistent. For the six non-winter events, the Penn State Runoff Model (PSRM) generated higher flows than those actually recorded ranging from approximately 90% higher to a factor of 12! (For the September 1985 event, PSRM produced a peak flow of 11,300 cfs compared to 923 cfs recorded at the gage.) For the two winter storms, however, PSRM under-predicted the recorded peak flows by 30 to 40 percent.

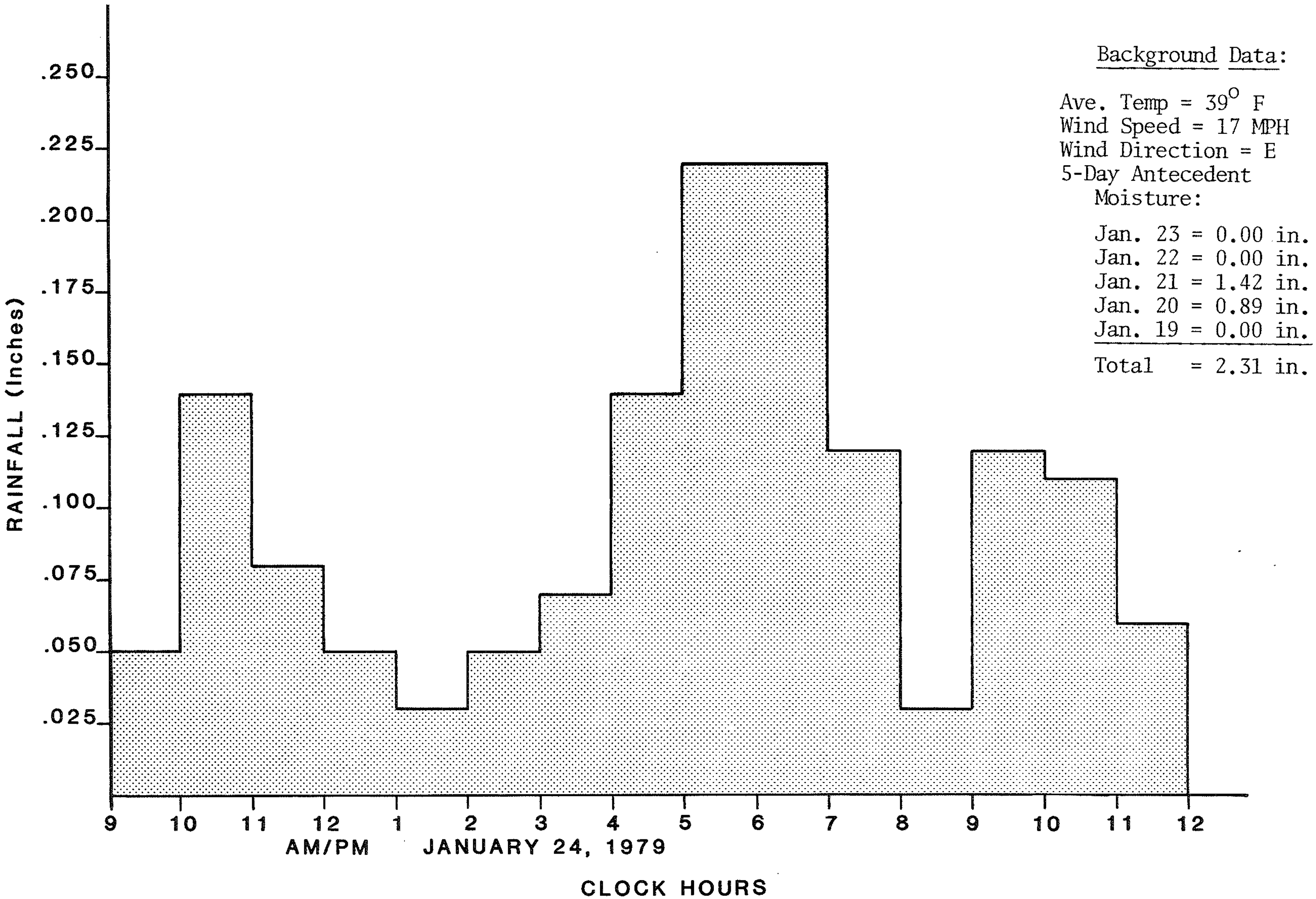
Presented in Figures 4 and 5 are the rainfall and runoff data, respectively, for the January, 1979 event which was the most severe runoff event of record. Figure 4 is called a hyetograph and indicates rainfall versus time in one-hour increments. Figure 5 is called a hydrograph and indicates flow versus time. The actual recorded peak flow for the Monocacy Creek was 3,490 cfs. From Table 2, the January, 1979 runoff event had a return period of approximately seventy (70) years.

Attempted calibration of the model using specific historical events was hampered by some apparent inconsistencies in the rainfall data. For the January 1979 storm, which produced 3,490 cfs from 1.51 inches of rain, PSRM was adjusted to represent completely developed conditions throughout the watershed and still could not reproduce the actual peak flow. Further, adjustments geared to reduce the PSRM 11,300 cfs peak for the September 1985 storm to the actual recorded peak of 923 cfs would have to go far beyond reasonable calibration adjustments. It is quite possible that the rain which actually fell on the watershed did not match the National Weather Service records. The NWS raingage is located approximately one-half mile west of the watershed and there are no recording raingages within the watershed itself.

Because of these factors, it was decided to calibrate the model using a design storm methodology. Specifically, the rainfall depth associated with a storm which would be expected to occur once every two years, on the average, can be determined by statistical analysis of rainfall data. Likewise, storm depths associated with return periods of 10-, 25-, 50- and 100-years can be determined. Calibration of the model can proceed on the assumption that the 2-year rainfall depth should produce approximately the 2-year streamflow. The hydrologic model was run for each of the 2-, 10, 25-, 50- and 100-year storms and systematic adjustments to model input were made to best approximate desired flow values. Adjustments made to the model included overland flow length and slope given the limestone geology, out-of-bank flow velocity and input of flow "losses" representing the process of streamflow being diverted to groundwater through solution channels in the limestone. Presented in Table 3 are the final calibrated peak flow values for the Monocacy Creek at the Monocacy Park gaging station for the various return period events.

FIGURE 4

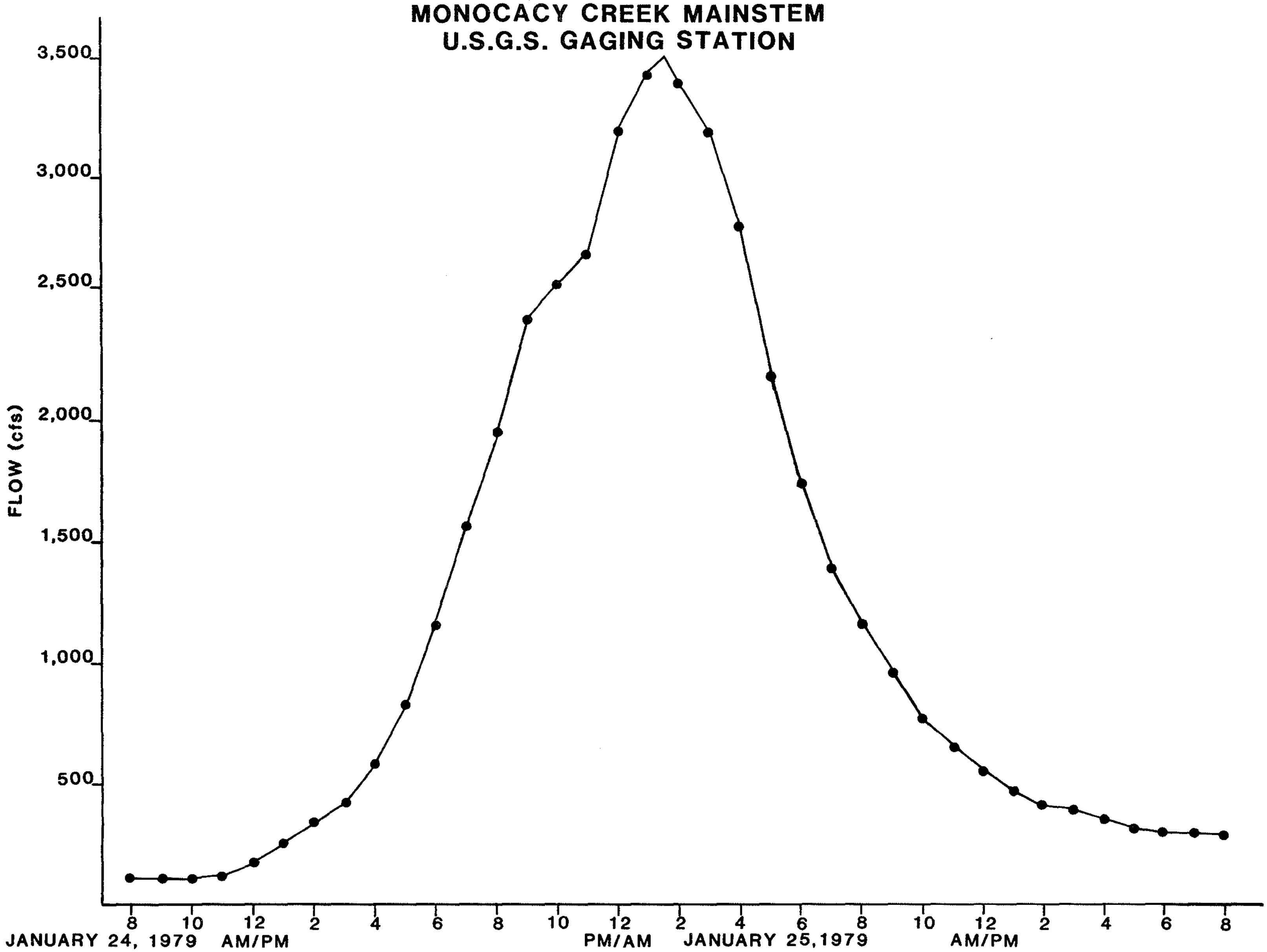
JANUARY 24, 1979 RAINFALL EVENT*



*Source: National Weather Service, A.B.E. Airport, Rainfall Records.

FIGURE 5

JANUARY 24-25,1979 RUNOFF HYDROGRAPH*
MONOCACY CREEK MAINSTEM
U.S.G.S. GAGING STATION



*Source: U.S.G.S. Gaging Station Records.

TABLE 3

Calibrated PSRM Versus Monocacy Creek Gaging Station Data at Monocacy Park Gage

Return Period	Peak Flow, cfs Gage Data	PSRM	PSRM % Difference
2	574	590	+ 3
10	1,656	2,033	+23
25	2,529	2,860	+13
50	3,361	3,989	+19
100	4,369	5,259	+20

Calibration of the runoff model is important for ensuring that the assumptions used in the physical data preparation and those inherent to the model itself do not preclude a reasonable representation of the actual runoff response of the watershed. It would not be reasonable to assume, however, that simply by calibration of the model that the flow rate at any point in the watershed could be defined accurately. Calibration of the model only deals with the runoff at the gaging station location. A calibrated model is simply a verification that the process used to develop the model is valid and that flows generated throughout the watershed can be used with an increased level of confidence.

CHAPTER III: MONOCACY CREEK WATERSHED LAND DEVELOPMENT AND RUNOFF IMPACTS

A. General Land Development Impacts on Storm Runoff

The necessity for the preparation of a storm water management plan is created by the fact that land development will, in general, cause a higher percentage of a given rainfall to become runoff. The primary reason for this is the increase in the amount of impervious cover on the land surface, i.e., roofs, driveways, parking areas, roads, etc.. Impervious cover does not allow rainfall to infiltrate into the ground. Therefore, rainfall which lands on impervious cover predominantly becomes runoff. The exception to this would be where impervious cover drains onto pervious areas which would provide for some infiltration. The percentage of impervious cover for a given development varies by the type of development. Table 4 below presents the "typical" percent imperviousness associated with the thirteen land use categories considered in this plan.

TABLE 4

"Typical" Percent Imperviousness by Land Use

	Land Use	Percent Imperviousness
1.	Woods	0
2.	Open Space	0
3.	Agriculture	0
4.	Low Density Residential	20
5.	Medium Density Residential	38
6.	High Density Residential	65
7.	Industrial	65
8.	Commercial	72
9.	Institutional	40
10.	Large Impervious Areas	100
11.	Water Bodies	100
12.	Transportation Uses	30
13.	Mining	0

The above typical percent imperviousness figures have been developed from standard Soil Conservation Service methodology. The breakdown between the three residential densities is as follows: low density - less than or equal to 2 units per acre; medium density - between 2 and 5 units per acre; high density - greater than or equal to 5 units per acre.

From Table 4 it is clear that the development of land which currently is in woods, open space, or agriculture could have a dramatic impact on the percentage of impervious cover. It is also clear that the cumulative impact of this type of development for a large watershed like the Monocacy Creek could be severe without implementation of the proper runoff management controls.

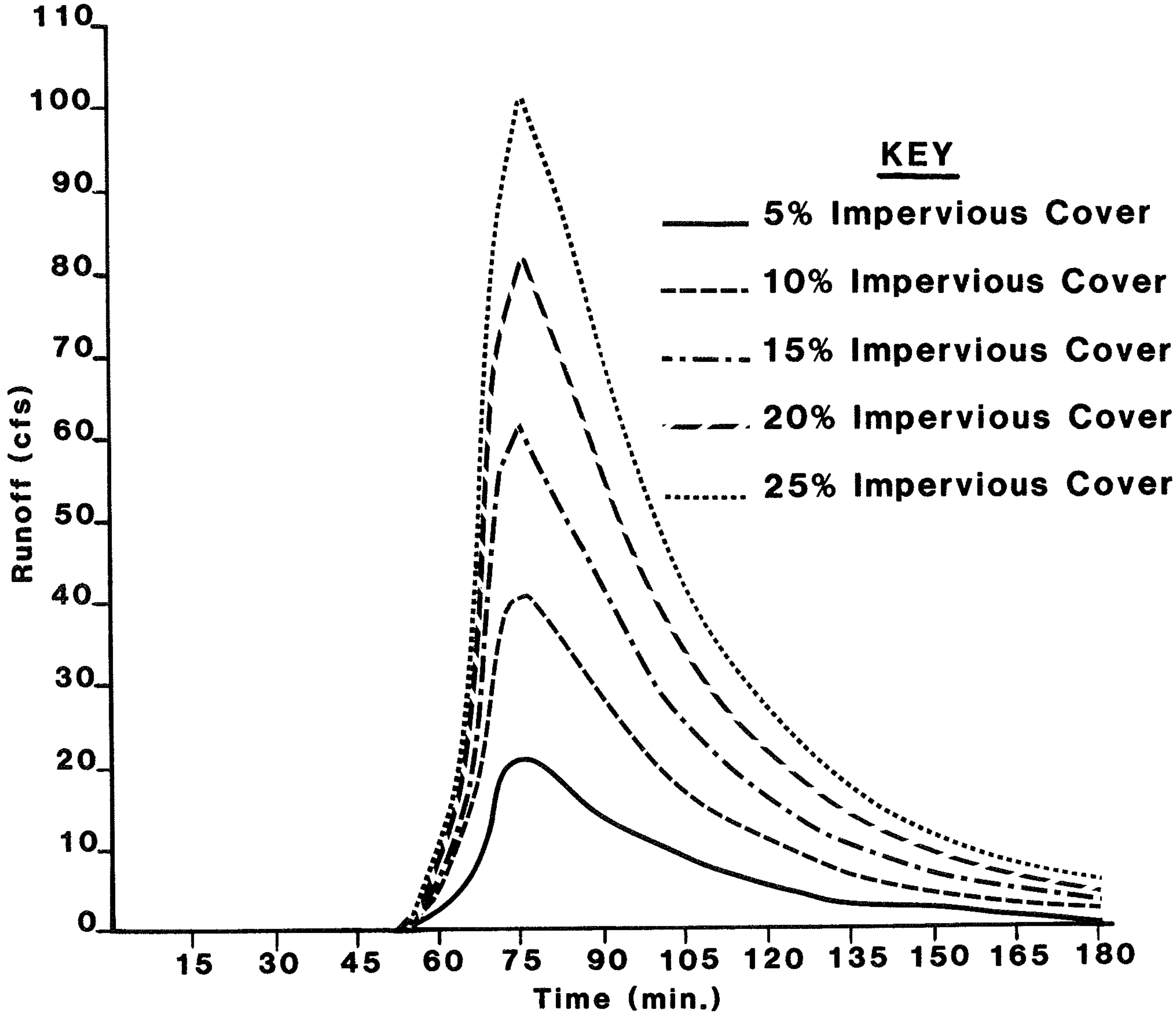
An example of the impact of increases in the amount of impervious cover for a given watershed area is illustrated in Figure 6. The series of curves, or hydrographs, present the runoff response of the watershed area versus time for percent imperviousness ranging from 5% to 25% as generated using the Penn State Runoff Model (the hydrologic computer model selected for analysis of the Monocacy Creek Watershed). The watershed area used for the analysis represents an average size subarea as used in the Monocacy Creek runoff modeling process (i.e. 300 acres). The rainfall event used to produce the hydrographs was a two-hour storm of 1.3 inch depth.

From Figure 6, the peak runoff from the watershed area for 5% impervious cover is approximately 20 cfs (cubic feet per second). Further, each 5% increment in impervious cover produces an additional 20 cfs to the peak runoff such that 25% imperviousness produces 100 cfs peak runoff. If the 5% impervious cover hydrograph represented the "existing" condition of a watershed area, then each 5% increment of impervious cover would produce a 100% increase in the predevelopment peak flow. In the Monocacy Creek Basin approximately half (51 out of 101) of the watershed subareas (as delineated for modeling purposes) have existing impervious cover of 5% or less. Again, it is clear that the runoff impacts of development of these subareas could be severe.

The amount of impervious cover is not the only factor affecting the amount of runoff produced by a given land area. Irrespective of impervious cover, certain land uses produce more runoff than others for the same rainfall. The Soil Conservation Service has researched the runoff response of various types of land uses, or land cover, and translated the results into a parameter called the runoff curve number. Simply described, the runoff curve number system is a ranking of the relative ability of various land use/cover types to produce runoff. Presented in Table 5 are the runoff curve numbers derived from SCS which have been used in the Monocacy Creek planning process. Higher curve numbers reflect a greater potential for producing runoff.

FIGURE 6

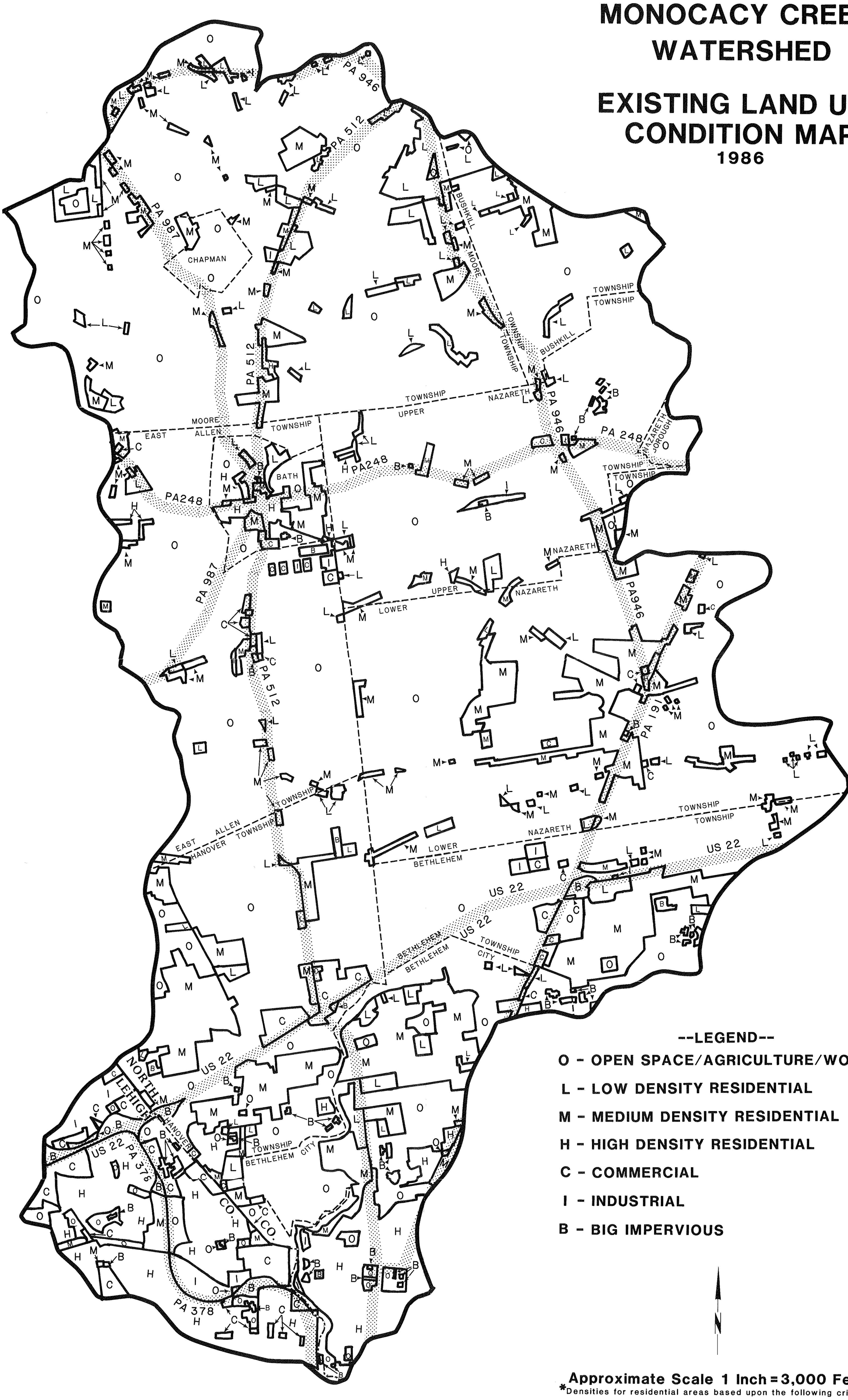
IMPACT OF IMPERVIOUS COVER ON STORM RUNOFF



MONOCACY CREEK WATERSHED

EXISTING LAND USE CONDITION MAP*

1986



--LEGEND--

O - OPEN SPACE/AGRICULTURE/WOODS

L - LOW DENSITY RESIDENTIAL

M - MEDIUM DENSITY RESIDENTIAL

H - HIGH DENSITY RESIDENTIAL

C - COMMERCIAL

I - INDUSTRIAL

B - BIG IMPERVIOUS

Approximate Scale 1 Inch = 3,000 Feet

*Densities for residential areas based upon the following criteria:

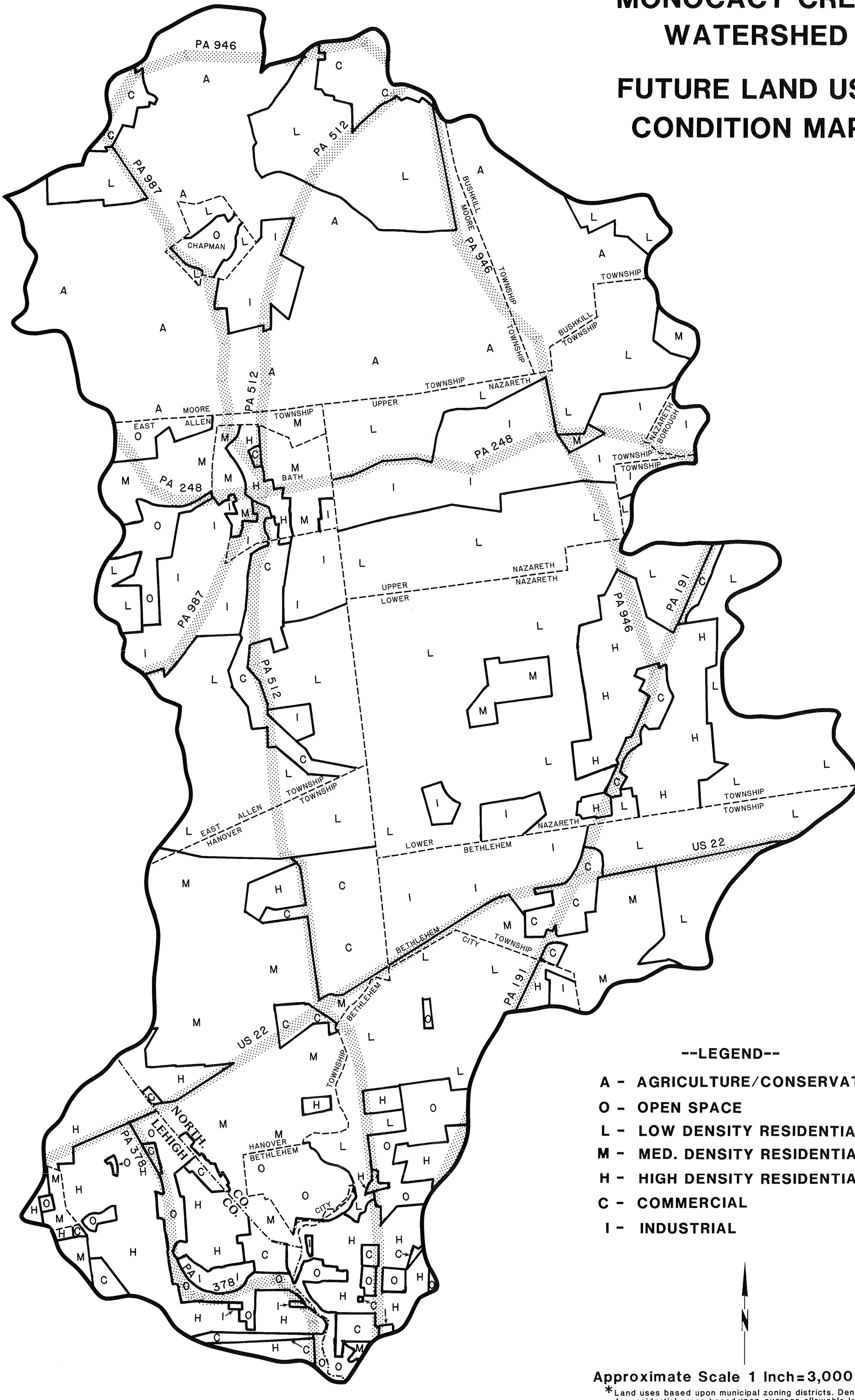
Low Density= <2 units/acre

Medium Density = 2 - 5 units/acre

High Density =>5 units/acre

MONOCACY CREEK WATERSHED

FUTURE LAND USE CONDITION MAP*



--LEGEND--

A - AGRICULTURE/CONSERVATION
O - OPEN SPACE
L - LOW DENSITY RESIDENTIAL
M - MED. DENSITY RESIDENTIAL
H - HIGH DENSITY RESIDENTIAL
C - COMMERCIAL
I - INDUSTRIAL

N

Approximate Scale 1 Inch=3,000 Feet

*Land uses based upon municipal zoning districts. Densities for residential areas based upon average allowable lot sizes and the following criteria: Low Density=<2 units/acre
Medium Density= 2-5 units/acre
High Density=>5 units/acre

TABLE 5

Runoff Curve Number by Land Use Category*

	Land Use	Runoff Curve Number
1.	Woods	55
2.	Open Space	61
3.	Agriculture	76
4.	Low Density Residential	68**
5.	Medium Density Residential	75**
6.	High Density Residential	85**
7.	Industrial	83**
8.	Commercial	87**
9.	Institutional	76**
10.	Large Impervious Areas	98**
11.	Water Bodies	100
12.	Transportation Uses	72**
13.	Mining	0

*Data is for Hydrologic Soil Group B.
**Curve Numbers reflect impervious cover percentages from Table 4.

Note from Table 5 that woods and open space have the lowest two curve numbers at 55 and 61, respectively, and both have zero percent impervious cover associated with them (from Table 4). Agriculture, however, even though it also has zero percent impervious cover, has a higher runoff curve number than both low and medium density residential land uses, which have 20% and 38% impervious cover, respectively.

It is not necessarily true from the above that agriculture will produce more runoff than low or medium density residential development and, in fact, agriculture can produce significantly less runoff than either one. Factors which affect this relationship are the slope of the land, the average length of overland flow, the rainfall event and the method of computation, among others.

One final factor affecting the impact of development on storm runoff is difficult to quantify, but perhaps very important in the Monocacy Creek Basin. The carbonate geology underlying approximately sixty-five percent (65%) of the basin has the characteristic of developing solution channels in the bedrock which can be manifested on the land surface as closed depressions and sinkholes. In the "existing" condition, the closed depressions and sinkholes can prevent a significant amount of runoff from entering the stream channel. Closed depressions simply create ponds of water and sinkholes divert surface runoff to the groundwater regime. The obliteration of these depressions and sinkholes with development can increase the storm runoff received by the stream beyond that anticipated using the curve number and percent impervious methodology.

The above-described impacts of development on storm runoff-impervious cover modification, curve number modification and removal of closed depressions - all relate to the rate and volume of runoff generated from a land area. An additional potential impact of development, however, is in the manner in which the generated runoff is conveyed downstream. Associated with development may be the construction of a closed pipe system to convey the runoff or the encroachment of the development into the natural conveyance channel, or both. Closed-pipe systems typically convey water faster than natural systems such that runoff is transported more quickly downstream. In addition, closed systems do not provide any opportunity for infiltration that exists with natural channels. Encroachment into the natural channel with development could take the form of fill on one or both sides, placement of structures or other modifications of the natural cross-section of the channel. The exact impact on the conveyance characteristics (i.e. depth, width, capacity, velocity) of the channel would depend on the type and extent of encroachment. A key aspect of the watershed plan is the ability of the conveyance facilities to maintain (or attain) adequacy for transporting anticipated runoff. Any modifications to the conveyance network associated with development should be accomplished in such a way as to best provide for continuing transport of upstream flows in a safe and efficient manner.

B. Historical Monocacy Creek Watershed Development

Development within the Monocacy Creek Watershed during the 1970's could be described as a concentration of urban-type development in and around the City of Bethlehem and more suburban development progressing north through the watershed. Bethlehem, Moore and Lower Nazareth Townships were the three most rapidly developing municipalities within the watershed throughout the seventies. Presented in Table 6 is the land development in acres for each municipality in the Monocacy Creek Watershed for the period of 1972 through 1986. The land development is broken down into residential, commercial and industrial land use categories. Data for the table has been obtained from JPC land use records. Note that for municipalities which are not completely within the watershed, the land development acres shown represent approximate values. From Table 6, over 1,700 acres of land were developed in the Monocacy Creek Watershed between 1972 and 1986 of which nearly 70% has been residential, 20% industrial and the remainder commercial. The 1,700 acres represent approximately 5% of the total watershed land area. For residential development, the acreages can be somewhat

Table 6

Monocacy Creek Watershed
Land Development by
Zoning Category 1972-1986*
(Acres)

Municipality	Residential	Commercial	Industrial	Total
Bath	9.2	4.9	3.7	17.8
Bethlehem (City)**	165.0	3.1	43.1	211.2
Bethlehem (Twp)**	178.1	64.9	8.7	251.7
Bushkill**	37.4	.0	.0	37.4
Chapman	6.7	.0	.0	6.7
East Allen**	36.2	48.2	85.5	169.9
Hanover (Lehigh)**	11.1	22.6	.0	33.7
Hanover (North.)**	120.0	73.4	7.8	201.2
Lower Nazareth**	208.6	11.7	126.8	347.0
Moore**	391.9	.6	2.3	394.8
Upper Nazareth**	16.8	10.6	60.0	87.3
Totals	1180.9	239.9	337.9	1758.7

*Source: JPC Records
**Represent approximated development figures for municipalities with significant areas outside of the Monocacy Creek Watershed.

misleading in that the density of development may vary significantly between municipalities. The number of units constructed in a given municipality could be disproportionate to the acreage when compared with another municipality.

Development in place in 1986 represents the "existing" situation insofar as the preparation of the watershed plan is concerned. The existing land use condition has been documented through review of land use records and through field surveys. A generalized existing land use map is included in the map jacket at the end of this chapter. Storm water runoff calculated based on the existing land use condition defines the goal of the watershed plan, i.e. no increase in existing peak flows throughout the watershed. The "stress" applied to the system is the increase in impervious cover in the watershed associated with new land development. Quantification of the stress requires an assumption of a future land use condition throughout the watershed. Future land use condition assumptions used in the development of the watershed plan are discussed in the following section.

C. Future Monocacy Creek Watershed Development

Projection of a future land use condition for the purpose of determining the runoff impacts of new development is an essential part of the plan preparation process. Only through an understanding of the increase in both volume of runoff and peak rate of runoff associated with development of the watershed can a sound control strategy be devised. Typically, a future land use condition is identified for a given "design year". The design year would be selected based upon the intended design life of the control strategy. Prudent storm water management would appear to dictate a design life consistent with full development of the watershed. Otherwise, the storm water management controls put in place today might quickly become outdated should development exceed expectations. Conversely, designing a runoff control strategy based upon the "ultimate" land use condition when that level of development may not occur for 10, 20 or even 40 years or more might appear somewhat impractical.

In an effort to help establish the merits of each approach, two future land use conditions, or scenarios, were investigated. The first is a design life-type scenario of estimating the anticipated development for a ten-year period (1987-1996). The second is a form of "ultimate" future land use based upon current zoning. Each of the scenarios is described below.

The land development projected over the period 1987-1996 was based on the continuation of the historical development trend. Rather than use the 1972-1986 time period as the basis of projection from Table 6, only the period of 1981-1986 will be used. Population growth figures from the 1980 Census and subsequent estimates verify the moderation of the growth trend.

Presented in Table 7 are the land development figures, in acres, for each municipality in the Monocacy Creek Watershed for the period of 1981-1986. The figures are presented for industrial (including warehousing), commercial and low-, medium- and high-density residential land use categories. For residential land use, the density breakdown is as follows: low density = 2 or less units per acre; medium density = between 2 and 5 units per acre; high density = 5 or more units per acre. From Table 7, Moore, Lower Nazareth and Bethlehem Townships dominated the development within the watershed over the five-year period. Each of the three municipalities had slightly over 100 acres developed, or 20-25% of the 596 total acres developed in the watershed. Of the three municipalities, Moore Township experienced the greatest increase with 138.8 acres, nearly all (98%) of which was low-density residential.

The projected land development figures for the period 1987-1996 are presented in Table 8. These projections in general represent continuation of the 1981-1986 trend for each municipality and each development category. The historical trend projection was modified in certain instances where knowledge of existing development proposals contradicted the historical development rates. The table indicates that approximately 995 acres would be developed over the next ten years.

Table 8 may provide a very reasonable estimate of the Monocacy Creek Watershed growth over the next decade. For storm water runoff purposes, however, it has a critical missing element. That is, within a given municipality, the table does not help identify where the growth may occur. As will be discussed in greater detail in subsequent chapters, the runoff control criteria will be developed for very small individual watershed areas of approximately 300 acres average size. Obviously, when considering watershed areas this small, the "where" question becomes important. An exaggerated example would be that the 137 acres of low-density residential growth listed for Moore Township could occur in scattered fashion throughout residentially-zoned areas (i.e. scattered watershed areas) or could be concentrated in only one or two of the 300 acre areas. The runoff control strategy devised to deal with these two situations could be very different.

The second future land use scenario evaluated is based on the assumption that development would occur throughout the

TABLE 7

Monocacy Creek Watershed
Land Development
1981-1986*
(Acres)

Municipality	Industrial	Commercial	Residential Low	Residential Medium	Residential High	Total
Bath	5.6	.0	-13.5***	1.8	2.9	-5.0
Bethlehem (City)**	.3	-2.3	23.1	12.0	11.9	44.9
Bethlehem (Twp)**	12.0	25.9	18.4	44.4	1.6	102.2
Bushkill**	.0	.0	15.1	.1	.0	15.2
Chapman	.0	.0	.8	.0	.0	.8
East Allen**	53.9	12.6	11.3	.5	.2	78.5
Hanover (Lehigh)**	.0	18.2	-.3	.4	.0	18.3
Hanover (North.)**	7.8	43.7	9.2	16.3	.0	77.0
Lower Nazareth**	61.2	9.6	36.4	6.5	.2	113.9
Moore**	1.8	.2	136.5	.2	.1	138.8
Upper Nazareth**	.0	5.3	3.6	.5	.0	9.4
Total	142.6	113.2	240.6	80.9	16.8	594.1

*Source: JPC Records
**Represents approximated development figures for municipalities with significant areas outside of the Monocacy Creek Watershed.
***Negative development figures indicate that the land in that category has been converted to other types of development.

TABLE 8

Monocacy Creek Watershed
Projected Land Development
1987-1996
(Acres)

Municipality	Industrial	Commercial	Low	Residential Medium	High	Total
Bath	9.4	.0	-22.5**	3.01	4.8	-5.3
Bethlehem (City)*	.5	-3.8	38.5	20.0	19.8	75.0
Bethlehem (Twp.)*	20.0	43.2	30.7	74.1	2.6	170.7
Bushkill*	.0	.0	25.3	.1	.0	25.4
Chapman	.0	.0	1.3	.0	.0	1.3
East Allen*	90.0	21.0	18.9	.8	.4	131.0
Hanover (Lehigh)*	.0	30.4	-.6	.6	.0	30.5
Hanover (North.)*	13.0	73.0	15.3	27.2	.0	128.5
Lower Nazareth*	102.2	16.1	60.8	10.8	.4	190.3
Moore*	3.0	.3	227.9	.3	.2	231.7
Upper Nazareth*	.0	8.9	6.0	.9	.0	15.7
Total	238.2	189.0	401.6	137.9	28.2	994.9

*Represent approximated development figures for municipalities with significant areas outside of the Monocacy Creek Watershed.
**Negative development figures represent continuation of the conversion of land to other types of development.

watershed based upon current zoning. Municipal zoning districts throughout the Monocacy Creek Basin can be categorized as industrial, commercial, agricultural or residential at various densities. For the purpose of evaluation of the future zoned condition land use, a composite zoning map of the watershed was prepared. Each of the zoning districts was placed into one of the above categories. The density criteria for residential development were the same as those used in the development of Tables 7 and 8. Since the allowable density of residential development can vary widely within a given zoning district, an "average" allowable density was determined from the district description and the district placed into a low-, medium- or high-density classification. The composite zoning map of the watershed was color-coded to reflect the categorization.

The future zoned condition land use map represents an "average ultimate" development scenario. It is an ultimate condition because all non-agriculturally zoned areas of the watershed are assumed to be developed. It is an average condition because, within a zoning district and consistent with the district description, development could occur at a higher or lower density than that assumed.

The decision regarding which of the two future land use conditions to use in structuring the runoff control philosophy can be made fairly readily when considering the structure of Act 167. The Act is based on the assumption that land development will continue to occur and that the storm runoff impacts associated with that development are to be controlled, but not that the development itself is to be controlled in location or rate or density. Using the 10-year design period development data would require assumptions as to the distribution of the development within the municipalities. The assumed distributions could be based upon concentrated development (perhaps adjacent to sewer lines) or based upon uniform scattered development. In either case, the accuracy of the development location assumptions for small watershed areas could suffer dramatically with unanticipated development in a very short period of time. Conversely, the future zoned condition land use would remain valid until either the zoning changed or major exception uses were allowed. Therefore, the future zoned condition land use will be used as the design land use for formulation of the runoff control plan. A map of the future land use condition as used in the development of the runoff control strategy is included in the map jacket opposite this page.

CHAPTER IV. FLOODPLAIN INFORMATION

A. Floodplain Delineation

The U.S. Department of Housing and Urban Development - Federal Insurance Administration has prepared a Flood Insurance Study for each municipality in the Monocacy Creek Basin. Seven (7) of these studies have been detailed investigations of the hydrology of the watersheds within the municipal boundaries including flood profiles (depth of water relative to channel elevations) and detailed mapping of floodplain areas. The remainder of the studies have been preliminary investigations of flood prone areas. Collectively, these studies document the 100-year floodplain within the Monocacy Creek Watershed. Each of the floodplain studies is available for inspection at the Joint Planning Commission offices as well as the respective municipal offices and is not reproduced here. A list of all the municipal Flood Insurance Studies including their date of preparation and whether they represent a detailed or preliminary study is presented in Table 9.

TABLE 9

Monocacy Creek Watershed
Flood Insurance Study Data

Municipality	Date of Study	Type of Study
Bath	February 17, 1988	Detailed
Bethlehem City	July 3, 1978	Detailed
Bethlehem Township	June 4, 1980	Detailed
Bushkill	March 4, 1988	Detailed
Chapman	July 30, 1982	Preliminary
East Allen	February 11, 1983	Preliminary
Hanover (Lehigh)	June 25, 1976	Preliminary
Hanover (Northampton)	August 1, 1977	Detailed
Lower Nazareth	May 4, 1988	Detailed
Moore	October 17, 1978	Detailed
Upper Nazareth	February 25, 1983	Preliminary

B. Existing and Future Floodplain Development

Currently within the Monocacy Creek floodplain the land use consists primarily of agriculture, open space, and low-density residential development. Located within the floodplain are various park properties including Keystone Park, Monocacy Park, Johnston Park, the Lower Nazareth Rod and Gun Club, Hanover Municipal Complex, 18th Century Industrial Area, Bethlehem Historic District, Lower Nazareth

Township Park, Gertrude Fox Conservation Area, Archibald Johnson Conservation Area and an unnamed County park located near Routes 191 and 22. Presented in Figure 7 is a map of the existing parks and recreation areas within the Monocacy Creek floodplain. Also shown in Figure 7 are areas which have been designated as floodplain preservation areas in the JPC Recreation and Open Space Plan (June, 1980).

The above notwithstanding, there also currently exists many instances of development within the 100-year floodplain in the Monocacy Creek Watershed. In the upper reaches of the watershed, which is relatively rural, floodplain development takes the form of scattered residences and encroachments associated with road crossings. In the downstream urban areas the natural floodplain has, in many instances, been completely modified by development activities resulting in higher flood damage potential and lesser flood carrying capacity.

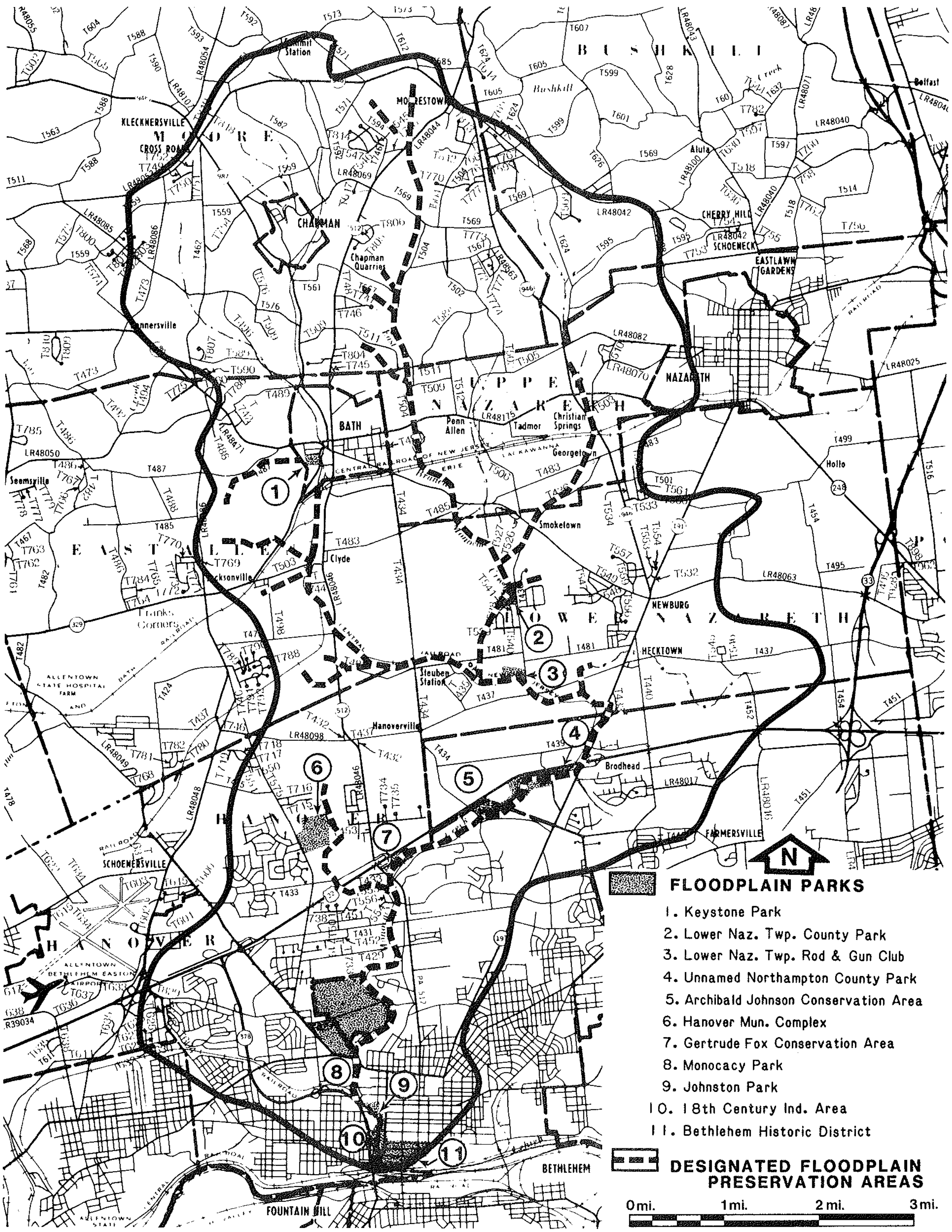
Development within the middle urbanizing areas of the watershed is taking place with a new set of rules that largely did not exist for the current urban areas. The new rules are those established by Pennsylvania Act 166 of 1978, the Floodplain Management Act. Act 166 required municipalities to adopt ordinances to regulate the type and extent of development within floodplain areas. All of the Northampton County municipalities in the watershed have enacted ordinances consistent with Act 166. With enforcement of those ordinances, any future floodplain development will be limited to that which would not significantly alter the carrying capacity of the floodplain or be subject to a high damage potential. A result of this has been that developments taking place adjacent to streams have had the floodplain areas dedicated for recreation and open space uses or otherwise been kept free of development.

For the purposes of the Monocacy Creek Storm Water Management Plan, the damage potential of existing and future floodplain development will be minimized using the following philosophy:

- Damage potential of existing floodplain development will remain unchanged for storm events representing the two-year through 100-year return period events through implementation of the storm water management criteria included in the Storm Water Management Plan for the Monocacy Creek Watershed.

- Damage potential for future floodplain development will be minimized by only permitting specific types of development which are damage resistant consistent with the Floodplain Management Act as implemented through municipal floodplain regulations and the Department of Environmental Resources Chapter 105 - Dam Safety and

FIGURE 7
MONOCACY CREEK FLOODPLAIN PARKS AND DESIGNATED FLOODPLAIN PRESERVATION AREAS



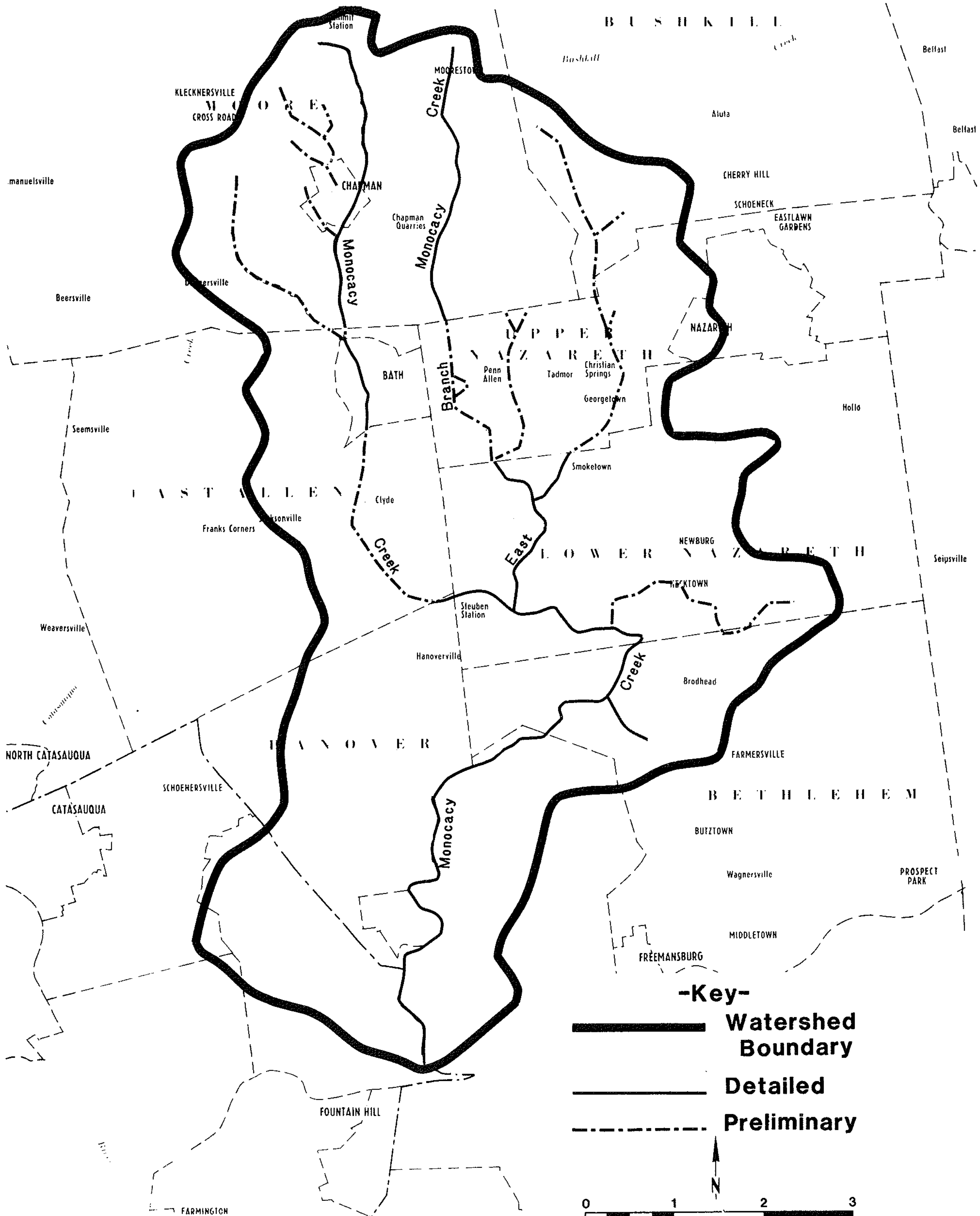
Waterway Management Regulations and Chapter 106 - Floodplain Management Regulations.

- o Damage potential of existing and future floodplain development may be reduced with implementation of remedial measures for areas subject to inundation. The effectiveness and design life of any remedial measures would be enhanced by implementation of the Storm Water Management Plan.

C. Detailed Versus Preliminary Floodplain Delineation by Stream Segment

Identified in Table 9 are the date and type of study of the Flood Insurance Studies for the Monocacy Creek Watershed municipalities. For those municipalities which have only preliminary flood studies, all stream segments within the municipality have had preliminary hydrologic investigations performed. For those municipalities which have had detailed flood studies performed, some stream segments have been evaluated in detail and others have only had preliminary investigations performed. Presented in Figure 8 is a map of the Monocacy Creek Watershed with the delineation of detailed versus preliminary flood investigations by stream segment.

FIGURE 8
DELINEATION OF DETAILED VERSUS PRELIMINARY FLOOD STUDIES BY STREAM SEGMENT



Source: FEMA Flood Insurance Studies

CHAPTER V. EXISTING STORM DRAINAGE PROBLEM AREAS AND SIGNIFICANT OBSTRUCTIONS

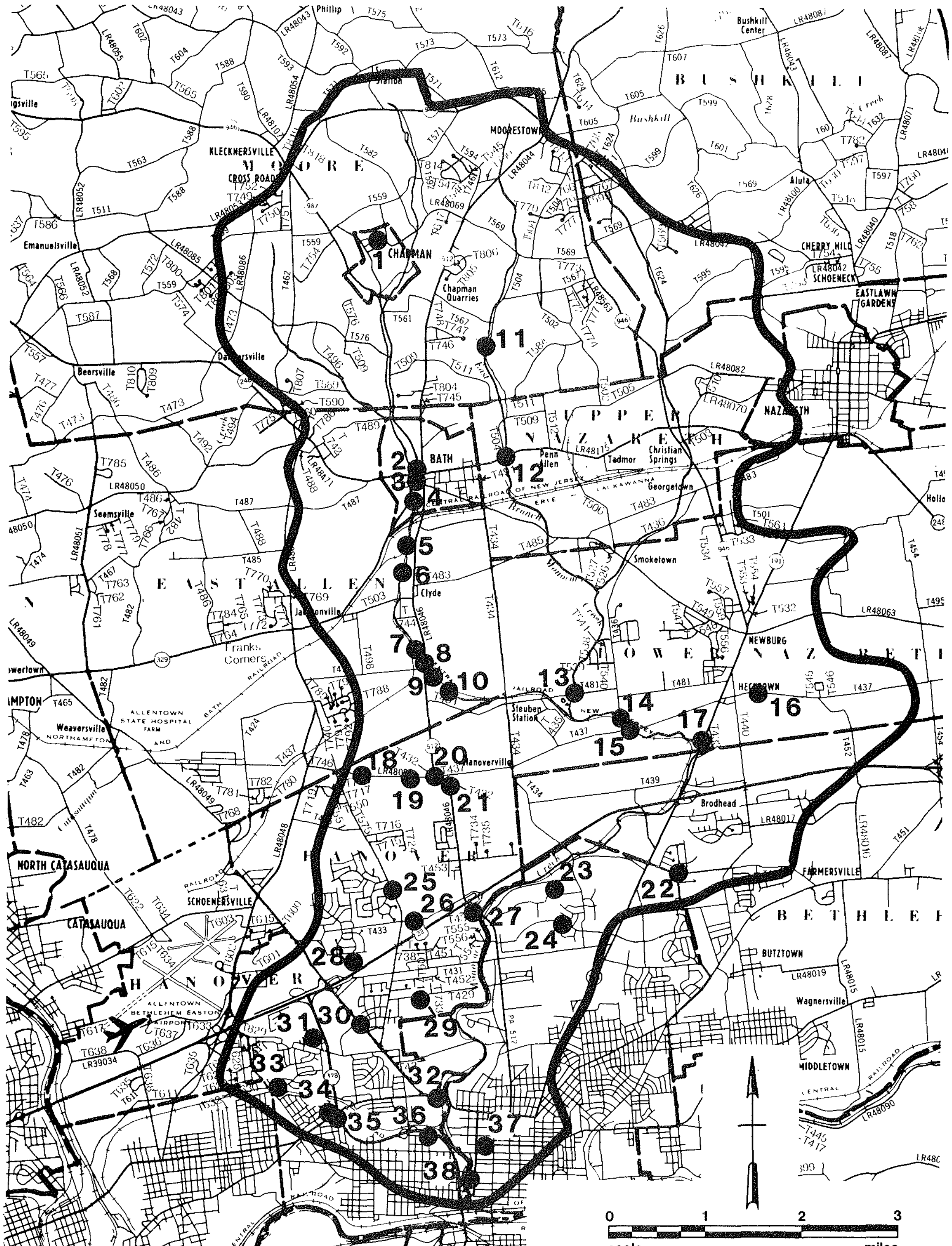
A. Existing Storm Drainage Problem Areas

An important goal of Act 167 is to prevent any existing storm drainage problem areas from getting worse. The first step toward that goal is to identify the existing problem areas. Each municipality in the Monocacy Creek Watershed was provided with an opportunity to document the existing drainage problems within its borders. The starting point for the drainage problem inventory was the JPC Regional Storm Drainage Plan (RSDP) which documented eight (8) problems throughout the watershed based on a municipal survey conducted prior to 1975. Each municipality had an opportunity to review the RSDP data, provide an updated status on whether the RSDP problems remained or had been corrected, and provide information on additional problem areas. This process resulted in the documentation of thirty-eight (38) existing drainage problems in the watershed. The type of problem identified was typically street and/or property flooding. There were fifty-two (52) problem areas documented in East Allen Township. Only six (6) of those involved flooding at streets or bridges, while the remaining forty-six (46) involve the build-up of debris along the mainstem not inducing specific flooding of a street or property. Therefore, only the six street or bridge related flooding problems were included in the inventory.

Figure 9 is a map of the Monocacy Creek Watershed indicating the storm drainage problem areas as identified as part of the Storm Water Management Plan. The problem areas on Figure 9 are number coded and keyed to the problem area descriptions presented in Table 10. The "subarea" and "Reach No." columns in Table 10 refer to the location of the problem areas relative to the watershed breakdown for modeling purposes. A subarea is the finest unit of breakdown of the watershed for which runoff values have been calculated. A reach is the swale, channel or stream segment which drains a particular subarea. Note that seventeen (17) of the drainage problems are on identified reaches indicating that peak runoff values are readily available from the modeling process for those problem areas. These runoff values could be used as input for design of remedial measures.

The final column in Table 10 lists generalized proposed solutions to the identified storm drainage problem areas. These generalized solutions have been provided by municipal representatives whether as part of the original problem area documentation or subsequent discussions. Proposed solutions listed include specific proposals based upon municipal studies of the problem areas, where available, and solutions which are readily apparent to the municipal representatives

FIGURE 9
MONOCACY CREEK WATERSHED
STORM DRAINAGE PROBLEM AREAS



See Table 10 for Number Identification.

Table 10

Storm Drainage Problem Areas

Number	Location	Municipality	Problem	Subarea	Reach No.	Proposed Solution*
1	Yost Road & 5th St.	Borough of Chapman	Street and Property Flooding	8	-	New Access Route for E. Branch
2	Northampton Street Bridge	Borough of Bath	Street and Property Flooding	18	15	None Proposed
3	Main Street	Borough of Bath	Street and Property Flooding	18	16	None Proposed
4	Mill Street	Borough of Bath	Street and Property Flooding	19	18	None Proposed
5	Railroad Bridge	East Allen Twp.	Property Flooding	19	18	None Proposed
6	Private Road	East Allen Twp.	Street Flooding	19	18	None Proposed
7	Railroad Bridge	East Allen Twp.	Property Flooding	20	19	None Proposed
8	Route 512	East Allen Twp.	Street Flooding	20&21	19	None Proposed
9	Railroad Bridge	East Allen Twp.	Property Flooding	21	20	None Proposed
10	Railroad Bridge	East Allen Twp.	Property Flooding	21	20	None Proposed
11	Keeler Road	Moore Township	Localized Flooding	36&37	36	May Have Been Solved by Larger Pipe Installment
12	Route 248 Near Penn Dixie Cement Company	Upper Nazareth	Street Flooding	43	42	Installation of Diversion Greatly Reduced Problem

*Proposed solutions are generalized solutions as provided by municipal representatives either based upon specific studies of the problems or their knowledge of the particular circumstances.

Table 10 (Cont'd)

Storm Drainage Problem Areas

Number	Location	Municipality	Problem	Subarea	Reach No.	Proposed Solution*
13	Steuben Road	Lower Nazareth Twp.	Street Flooding	62&63	62	None Proposed
14	Georgetown Road	Lower Nazareth Twp.	Street Flooding	64	63	None Proposed
15	Hanoverville Road	Lower Nazareth Twp.	Street Flooding	66	64	None Proposed
16	Hecktown Road	Lower Nazareth Twp.	Street Flooding	67	-	None Proposed
17	PA Route 191	Lower Nazareth Twp.	Property Flooding	70	69	None Proposed
18	Culvert	Hanover Twp. (N)	Street Flooding	82	-	Cross Culverts
19	Culvert	Hanover Twp. (N)	Street Flooding	82	-	Cross Culverts
20	Culvert	Hanover Twp. (N)	Street Flooding	82	-	Cross Culverts
21	Route 512 at Ackerman's	Hanover Twp. (N)	Street Flooding	82	-	Shoulder Grading of Route 512
22	Oakland Road	Bethlehem Twp.	Street Flooding	76	-	None Proposed
23	Pine Top Trail/ Fox Dr./Bierys Bridge Road	City of Bethlehem	Property Flooding	79	-	Improve Channel Capacity
24	Johnston Drive	City of Bethlehem	Street Flooding	79	-	None Proposed

*Proposed solutions are generalized solutions as provided by municipal representatives either based upon specific studies of the problems or their knowledge of the particular circumstances.

Table 10 (Cont'd)

Storm Drainage Problem Areas

Number	Location	Municipality	Problem	Subarea	Reach No.	Proposed Solution*
25	Stafore Estates/ Gaspar Area	Hanover Twp. (N)	Street and Property Flooding	81	-	Install and Extend Pipes
26	Stoke Park Road Area	Hanover Twp. (N)	Street Flooding	85	-	Install Catch Basin/ Improve Existing Ones
27	Monocacy Creek at Rte. 512	Hanover Twp. (N)	Localized Floods	86	80	Box Culvert/Swale Construction
28	Hanover Farm Storm System	Hanover Twp. (N)	Street Flooding	93	-	Piped Collection/ Conveyance System
29	Bridle Path Road Area	Hanover Twp. (N)	Street Flooding	87	-	Additional Piping
30	Westgate Area	Hanover Twp. (N)	Street Flooding	95	-	Detention/Additional Inlets/Increased Conveyance Capacity
31	Valley Park South Apartments	City of Bethlehem	Property Flooding	96	-	Additional Detention Upstream
32	Schoenersville Rd.	City of Bethlehem	Street Flooding	92	91	None Proposed
33	Pinehurst Road	City of Bethlehem	Street and Property Flooding	97	-	Detention Facility
34	Homestead Avenue	City of Bethlehem	Street and Rear Yard Flooding	97	-	None Proposed

*Proposed solutions are generalized solutions as provided by municipal representatives either based upon specific studies of the problems or their knowledge of the particular circumstances.

Table 10 (Cont'd)

Storm Drainage Problem Areas

Number	Location	Municipality	Problem	Subarea	Reach No.	Proposed Solution*
35	Highland and Eaton Avenues	City of Bethlehem	Street and Property Flooding	98	-	None Proposed
36	5th Avenue at Rte. 378	City of Bethlehem	Property Flooding	98	-	Diversion of Runoff to Route 378
37	Goepp Street	City of Bethlehem	Street Flooding	100	-	Additional Inlets and Relief Pipe System
38	Historical Bethlehem Tannery Building	City of Bethlehem	Property Flooding	100	-	None Proposed

*Proposed solutions are generalized solutions as provided by municipal representatives either based upon specific studies of the problems or their knowledge of the particular circumstances.

for the less complicated problem areas. For certain other problem areas, the solutions are not quite so apparent and may require detailed engineering evaluations to determine the most cost-effective solution. No solutions to these problem areas are available and are listed as "None proposed" in Table 10.

B. Significant Obstructions

An obstruction in a watercourse can be defined borrowing from Chapter 105 of DER's Rules and Regulations as follows:

> "Any dike, bridge, culvert, wall, wingwall, fill, pier, wharf, embankment, abutment or other structure located in, along, or across or projecting into any ... channel or conveyance of surface water having defined bed and banks, whether natural or artificial, with perennial or intermittant flow."

For the purpose of Act 167, it is necessary to narrow the definition to include only those obstructions which are "significant" on a watershed basis. For the Monocacy Creek Storm Water Management Plan, the following distinction will be used:

> An obstruction in a stream or channel shall be deemed "significant" if it has been documented to create a flooding or backwater condition as identified in the municipal Flood Insurance Studies.

Using the above definition, sixteen significant obstructions have been identified within the Monocacy Creek Watershed and are shown in Figure 10. A list of the significant obstructions is presented in Table 11 which indicates the obstruction number, description, municipality and <u>approximate</u> flow capacity. Obstruction capacities have been estimated based on their upstream geometry as measured and bed slope and roughness factors (where applicable) consistent with the calibrated Penn State Runoff Model for the Monocacy Creek Watershed. The estimates reflect reasonable flow capacities of the obstructions for "open channel" flow conditions (i.e. where the obstructions are not submerged). These estimated capacities are for illustration only and shall not be used as absolute capacities for storm water management decisions. The capacity of any obstruction when used to meet the requirements of this Plan shall be based upon a detailed hydraulic investigation including possible headwater and tailwater conditions, obstruction configuration (abutments, wingwalls, piers, etc.) field measured slopes and other conditions as may affect capacity for design flows.

There are three (3) identified significant obstructions which coincide with documented storm drainage problem areas as

FIGURE 10
MONOCACY CREEK WATERSHED
SIGNIFICANT OBSTRUCTION INVENTORY FROM FIA* STUDIES

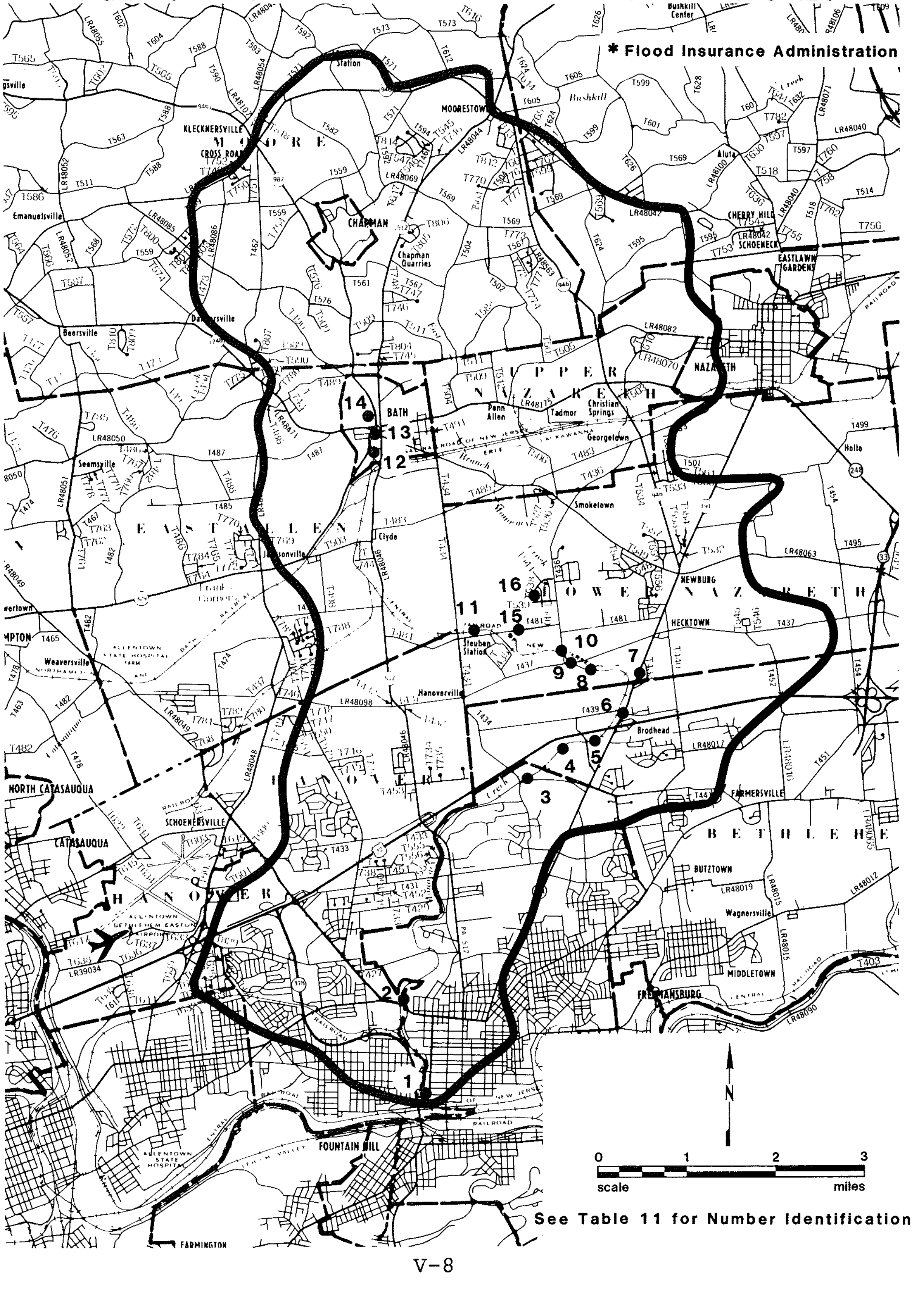


Table 11

Significant Obstruction Inventory from FIA* Studies

Number**	Obstruction	Municipality	Approximate Flow Capacity (cfs)***
	-MAIN BRANCH MONOCACY CREEK-		
1	Abandoned Lehigh Canal	City of Bethlehem	4,500
2	Tunnel Bridge	"	2,200
3	Conrail	"	1,800
4	Conrail Crossing #2	Bethlehem Township	1,600
5	Conrail Crossing #3	"	1,500
6	Broadhead Road	"	4,500
7	State Route 191	Lower Nazareth	50,000
8	Conrail	"	750
9	Conrail Bridge	"	1,100
10	Georgetown Road	"	2,200
11	Access Road	"	330
12	Mill Street[1]	Borough of Bath	2,100
13	Main Street/State Rt. 248[2]	"	1,300
14	Silk Street	"	N/A
	-EAST BRANCH MONOCACY CREEK-		
15	Steuben Road[3]	Lower Nazareth	670
16	Access Road	Lower Nazareth	1,700

*Flood Insurance Administration
**Numbers are keyed with Significant Obstruction Map (Figure 10).
***Estimated capacities are for illustration only and shall not be used as absolute capacities for storm water management decisions.

[1]Obstruction No. 12 coincides with Problem Area No. 4.
[2]Obstruction No. 13 coincides with Problem Area No. 3.
[3]Obstruction No. 15 coincides with Problem Area No. 13.

indicated in Table 11. Each obstruction which coincides with a drainage problem area is footnoted in Table 11 with the corresponding problem area number identified at the end of the table. The importance of the identified significant obstructions and problem areas as part of the development of a runoff control strategy is discussed in Chapter VIII.

CHAPTER VI. STORM RUNOFF CONTROL TECHNIQUES

Chapter III identified the impacts of land development on storm water runoff and documented the need to control those impacts with sound storm water management techniques. Chapter VIII presents the performance standards for runoff control for new development applicable to the various watershed areas necessary to achieve sound runoff management from a watershed perspective. Therefore, Chapter III defines the problem and Chapter VIII identifies the necessary end product. This chapter will identify the runoff control techniques available as the "means" to the desired end product to mitigate the runoff impacts of new development.

The runoff control techniques presented are "structural" storm water management controls meaning that they are physical facilities for runoff abatement. "Nonstructural" controls refer to land use management techniques geared toward minimizing storm runoff impacts through control of the type and extent of new development throughout the watershed. The Monocacy Creek Storm Water Management Plan is based on the assumption that new development of various types will occur throughout the watershed (except as regulated by floodplain regulations) and that structural controls will be required to minimize the runoff implications of the new development.

Structural controls for managing storm runoff can be categorized as either volume controls or rate controls. Volume controls are those which are designed to prevent a certain amount of the total rainfall from becoming runoff by providing an opportunity for the rainfall to infiltrate into the ground. Greater opportunity for infiltration can be provided by minimizing the amount of impervious cover associated with development, by draining impervious areas over lawns or other pervious areas or into specific infiltration devices, and by using grassed swales or channels to convey runoff in lieu of storm sewer systems. Rate controls are those which are designed to regulate the peak discharge of runoff by providing temporary storage of runoff which otherwise would leave the site at an unacceptable peak value. Rate controls, much more so than volume controls, are adaptable to regional considerations for controlling much larger watershed areas than one development site.

Presented below is a discussion of the various volume and rate controls available for implementation for a development site (or region). The discussion includes a physical description of the control, the applicability of the particular control, its advantages and disadvantages and maintenance requirements. The runoff control(s) most applicable to a development site may vary widely depending upon site characteristics such as topography, soils, geology, water table, etc., the type of development proposed and the applicable performance standard which the controls must meet. The developer should consider all these factors in designing the control philosophy.

The runoff control technique information presented herein has been derived primarily from two sources; namely, (1) New Jersey Stormwater Quantity/Quality Management Manual, February 1981, prepared for the N.J. Department of Environmental Protection by the Delaware Valley Regional Planning Commission, and (2) Allegheny County Act 167 Pilot Stormwater Management Plans - Girty's Run, Pine Creek, Deer Creek and Squaw Run, January 1982 prepared for the Allegheny County Department of Planning by Green International, Inc. and Walter B. Satterthwaite Assoc., Inc.

A. Volume Controls

The increase in runoff volume with development, and the management of that increased volume, is a key element in sound runoff management at the watershed level. Any volume controls implemented on-site for a development would help achieve the goals of the watershed plan. As stated above, the basis for volume controls is the provision of a greater opportunity for infiltration of rainfall/runoff into the ground. This opportunity may be provided in a passive sense by simply draining impervious areas over pervious areas and relying on the natural infiltrative capabilities of the pervious areas. Conversely, the opportunity for infiltration could be provided in an active sense by directing runoff into infiltration structures designed to remove a given volume of runoff. A different type of volume control is based upon the substitution of porous or semi-pervious materials in place of conventional impervious surfaces. Any or all of these approaches may be applicable to a particular development site.

Typically, volume controls would be used in conjunction with rate controls since volume controls themselves would not generally provide an adequate level of runoff abatement. The volume controls would, however, provide the benefit of decreasing the size and cost of the rate control facility and would be used to minimize the total cost of on-site runoff control.

The primary environmental benefit of volume controls is groundwater recharge. This is a worthy and desirable goal, but one which may create more problems than it solves if indiscriminately applied. The major reason for this is the limestone geology which dominates the lower Monocacy Creek Watershed. Characteristic of the limestone geology are solution channels in the bedrock which provide flow conduits for infiltrated rainfall and may be manifested on the land surface with closed depressions or sinkholes. The direct (or nearly direct) connection between surface water and groundwater could lead to discharge of polluted runoff into the groundwater regime without proper opportunity for pollutant removal by soils. In addition, any modification of the predevelopment drainage condition through the use of volume controls may increase the possibility of sinkhole occurrence. It is necessary, therefore, to conduct a thorough site evaluation in limestone areas to identify potential groundwater pollution or sinkhole prone areas and design the runoff control plan accordingly.

1. Infiltration Pits and Trenches

a) Description

Infiltration pits and trenches usually consist of excavated pits or trenches, backfilled with sand and/or graded aggregates, in which storm water runoff is collected for temporary storage and subsequent infiltration.

Infiltration pits vary in depth from about 6 to several hundred feet, depending upon the depth of the permeable soil strata and the depths to groundwater and bedrock. A "dry well" consists of a perforated structural chamber buried in the ground which is empty or filled with aggregates, depending upon the strength of the structure. Dry wells are commonly used to collect and infiltrate runoff from rooftops or other areas free of sediment and debris.

Infiltration trenches are long narrow excavations with depth normally less than 6 feet. Although a variety of geometries may be used, higher infiltration rates are usually attained from wide, shallow trenches. Where infiltration trenches are not protected by a grating, wheel stops or segmented curbs are necessary to keep off vehicular traffic.

b) Applicability

These controls may be used where the subsoil is sufficiently permeable to allow a reasonable rate of infiltration and where the water table is sufficiently lower than the design depth of the facility. Not applicable where high concentrations of suspended materials are contained in the runoff without some type of filtering mechanism.

c) Advantages and Disadvantages

ADVANTAGES

- Can be incorporated into the design of storm sewer systems to reduce the required flow capacity and cost, or to reduce overflow occurrences.
- May help reduce local flood peaks.
- Relatively inexpensive to construct.
- Utilizes existing natural drainage system.
- Groundwater recharge.

DISADVANTAGES

- Susceptible to clogging by sediment.
- Landscaping requirements may produce aesthetically objectionable conditions or safety hazards.
- Dry wells often require an emergency collection basin surrounding the beds since failure causes flooding.
- Maintenance is difficult when the facility becomes clogged.
- Limited in application to small sources of runoff such as roof drains, small parking lots, tennis courts, etc.

d) Maintenance Requirements

Preventive maintenance is vital to the continued effectiveness of infiltration facilities. Once void areas become clogged, maintenance entails a complete replacement of filter material. The use of filter fabrics over the surface of the facility is helpful, although periodic cleaning or replacement will be necessary. Runoff from roofs and grass covered areas or frequently cleaned parking lots can be stored and infiltrated with minimal problems. In areas where runoff is likely to carry considerable amounts of suspended materials, other measures should be considered.

2. Concrete Grid and Modular Pavement

a) Description

Pervious pavement systems consisting of strong structural materials containing void areas which can be filled with pervious materials such as sod, gravel, or sand. Categories include:

Poured-in-Place Slabs

Reinforced concrete slabs covering large areas are poured in-place on the ground to be covered. Special forms are used to shape the void areas, and a flat surface results. Because the slab is continually reinforced with steel, this pavement is suitable for heavy loads and has maximum resistance to movement caused by frost heave or settling.

Pre-Cast Concrete Grids

Concrete paving units incorporating void areas, usually precast in a concrete products plant and trucked to a job site for placement on the ground to be covered. However, for large jobs, these units can be formed and cast at the site.

Modular Unit Pavers

Smaller pavers which may be clay bricks, granite sets, or cast concrete of various shapes. These pavers do not have void areas incorporated into their configuration. They are installed on the ground with pervious material placed in the gaps between the units.

c) Advantages and Disadvantages

ADVANTAGES

- Flexibility - sections can be lifted to plant trees, place signs, maintain utility lines beneath.
- Can be used in some situations where porous asphalt is not suitable. For example, areas subject to sinking or heaving.
- Represents a compromise between a natural grass and an asphalt or concrete surface aesthetically, hydrologically, and quality-wise.

DISADVANTAGES

- Expensive and difficult to lay.
- Fertilizers and de-icing chemicals may have adverse effects on concrete products.
- Can present safety hazards.

d) Maintenance Requirements

Where turf is incorporated as the porous surface medium, normal turf maintenance (such as watering, fertilizing and mowing) will be necessary. Infrequent mowing is required in high traffic areas. However, use of fertilizers and de-icing chemicals should be restricted as much as possible. Because they are monolithic and maintain a smooth surface, poured-in-place installations can be snow-plowed provided damage to the grass cover can be avoided.

3. Porous Asphalt Pavement

a) Description

Porous asphaltic pavement material consists of a graded aggregate held together by (asphalt) cement and containing sufficient void space to allow a high rate of permeability to water. The nature of each individual site will influence the specific design of the porous pavement. Each design will depend upon soil conditions, expected surface wear, and the particular use of the surface.

b) Applicability

Most suitable for low-volume traffic areas such as parking areas, residential streets, recreation surfaces, airport runways and wherever subgrade soils have moderate permeability.

c) Advantages and Disadvantages

ADVANTAGES

- May reduce size of or eliminate additional drainage facilities. For instance, storm sewers, catch basins, curbs and gutters and so forth.
- Improved preservation of roadside vegetation.

- Flexible measure to provide storm water detention in both new and existing development.

- Safety improvements such as superior skid resistance during wet conditions and enhanced visibility of pavement markings.

- Provides pavement drainage without the need for a crown slope, thus reducing costs and puddling.

- Offers aesthetic alternatives since color selection is possible.

- Less noisy than conventional pavements.

- Less costly than conventional pavements for most applications.

- Enhances groundwater supply.

DISADVANTAGES

- Technique is relatively new with claims more founded on laboratory results rather than real-life experiences.

- Open-graded mixtures may be more prone to water stripping than conventional dense aggregate mixtures.

- Increased pressure head on pavement from subsurface drainage on steep slopes.

- Clogging may be a problem in some environments.

- Freezing and thawing may present problems although there is little evidence of this problem.

- Water that freezes within the porous pavement takes longer to thaw and offsets infiltration.

- Motor oil drippings and gasoline spillage may pollute groundwater.

d) Maintenance Requirements

Maintenance involves removal of debris too coarse to be washed through the pavement system; vacuuming to remove particles that could clog the void space; and patching the surface as needed. Since porous pavements require no additional repairs than conventional pavements, maintenance problems can be generally confined to better "housekeeping" and "preventive maintenance" practices and more efficient and effective street cleaning procedures in municipalities.

4. Grassed Waterways, Filter Strips, and Seepage Areas

a) Description

This practice utilizes grassed areas for managing storm water runoff by using their natural capacity for reducing runoff velocities, enhancing infiltration, and filtering runoff contaminants. Such measures include:

Grassed Waterways - Concentrated flows of surface runoff are directed through grass covered drainage swales or channels. The grassed surface retards flow velocities and maintains soil porosity while providing relatively stable channel lining. In addition, a small amount of runoff filtering occurs due to the velocity reduction, resulting in improved water quality. Whenever possible, grasses native to the site should be selected for use to insure acclimation.

Filter Strips - Sheet flows of surface runoff are directed across grass buffer strips which slows the sheet flow causing the heavier particulates to fall out while simultaneously enhancing infiltration of the runoff. These strips of close growing grasses can be established at the perimeter of disturbed or impervious areas.

Seepage Areas - Surface runoff is directed into small grass covered areas that infiltrate the water and filter out particulate contaminants. Seepage areas are created by excavating shallow depressions in the land surface or by constructing a system of dikes or berms to temporarily pond water over permeable soils.

b) Applicability

Mostly applicable in new developments of low to moderate density where the percentage of impervious cover is relatively small. These practices also require that subdivision and site designs respect natural drainage patterns so that they can be modified to accommodate post-development runoff volumes.

Successful application is dependent upon such factors as steepness of slopes, anticipated runoff volumes, soil conditions, selection of proper grass cover and proper long-term maintenance.

c) Advantages and Disadvantages

ADVANTAGES

- o Vegetative swales are less expensive to install than curb and gutter systems.
- o Roadside ditches keep flow away from the street thereby reducing the potential for hydroplaning.
- o Groundwater recharge.

DISADVANTAGES

- o Vegetative channels may require more maintenance than curb and gutter systems.
- o Streets with swales may require more right-of-way and be less compatible with sidewalk systems.
- o Proper selection of filter strip width is presently a judgmental decision.
- o Roadside ditches become less feasible as the number of driveway entrances requiring culverts increases.
- o Local subdivision ordinances may require curbs and gutters, so municipalities may have to amend their regulations to allow this practice.

d) Maintenance Requirements

<u>Grassed Waterways</u>

Periodic inspections, especially after large storms, are required to evaluate whether erosion controls are needed, to remove accumulated debris, and to check the condition of the vegetation.

<u>Filter Strips</u>

Like grassed waterways, periodic inspections are necessary but it is particularly important to maintain soil porosity. This can be accomplished by periodically removing thatch and/or mechanically aerating the area when necessary.

<u>Seepage Areas</u>

Similar maintenance considerations are required as for grassed waterways and filter strips.

B. Rate Controls

The performance standard criteria presented in Chapter VIII are geared towards controlling the peak rate of runoff after development to a given percentage of the predevelopment peak runoff rate. The bases for establishing the performance standards are the predevelopment peak rate, the timing of the predevelopment peak with respect to other watershed areas and the anticipated increase in volume associated with development. The volume controls described in Part A will remove a portion of the increased volume of runoff and may also help to reduce the peak rate of runoff. It is primarily the rate controls, however, which provide the major peak attenuation by storing a large volume of runoff and releasing it at a predetermined slower rate. The various options available for rate control differ only in the location of the runoff storage provided as described below.

1. Detention Basins

a) Description

Detention basins are impoundments which are designed to store "excess rate" storm water runoff during a rainfall event and release the stored runoff more slowly. "Excess rate" can be defined as the difference between the uncontrolled post-development hydrograph and the design post-development hydrograph as dictated by the performance standard criteria. Detention basins may be designed as either dry or wet impoundments. Dry impoundments are designed to completely drain after storm events. Wet impoundments are designed to maintain a permanent pool.

The storage volume required for a detention basin is a function of the change in runoff volume and the pre- and post- development peak, the performance standard applicable to the site, the extent to which volume controls are used, the outlet structure configuration and the design storm(s) used.

b) Applicability

Detention basins are applicable to any development site where rate control is required and sufficient land area exists. Detention basins can be designed for individual site control or to control runoff from multiple development sites or watershed areas.

c) Advantages and Disadvantages

ADVANTAGES

- o Offers design flexibility for adopting to a variety of uses.
- o Construction of ponds is relatively simple.
- o May allow significant reduction in the size of downstream storm drainage structures.
- o Enhances groundwater recharge to some degree.
- o Decreased runoff rate from pond reduces stream channel erosion.
- o Reduce downstream litter and debris.

DISADVANTAGES

- o Possible aesthetic and safety considerations.
- o Maintenance programs may present problems.
- o Consumes land area which cannot be developed.
- o In limestone geology, soil depth and type must be considered in design to minimize possibility of sinkhole occurrence.

d) Maintenance Requirements

In order to maintain the design efficiencies of a detention basin, maintenance of the structures and the impoundment areas are essential. To be effective, a formal maintenance plan should be formulated. Elements of such a plan should include:

- o Routine inspection of pipe inlets and outlets for accumulated sediment and debris.
- o Critical area stabilization and vegetative control.
- o Measures to offset the production of fast-breeding insects, as necessary.
- o Periodic inspection by a qualified professional engineer to ensure that impoundments remain structurally sound and hydraulically efficient, including evidence of possible sinkhole formation.

2. Parking Lot Storage

a) Description

Parking lot ponding is usually achieved by using specifically designed or modified inlet structures thereby causing temporary ponding in portions of a parking lot, generally at the perimeter, and specifically graded for that purpose. This technique is presently used in many municipalities to deal mainly with relatively small storm events.

b) Applicability

Where portions of large, paved parking lots can be temporarily used for storm water storage without significantly interfering with normal vehicle and pedestrian traffic. Shopping centers and large employee parking areas are likely places for use of this measure.

c) Advantages and Disadvantages

ADVANTAGES

- Can contribute to maintaining adequate capacity of downstream drainage facilities.
- Adaptable to both existing and new parking facilities.
- Parking lot storage is usually easy to incorporate into parking lot design and construction.

DISADVANTAGES

- May cause public inconvenience.
- Ponding areas are more prone to icing in cold weather.

d) Maintenance Requirements

Inspections should be performed periodically and following large storms in order to assure proper functioning.

3. Rooftop Detention

a) Description

Rooftop ponding makes use of the structural capabilities of rooftops to detain and release rainfall volumes such that flows are more gradually collected in sewers and streams. This effect is achieved through the use of small perforated weirs or collars placed around the inlets of roof downdrain pipes. When the water exceeds the designed pond depth, overflow occurs and the downdrains are allowed to function at peak capacity. The weirs are also designed such that no water is stored during small storm events. Experience with this practice has indicated that additional surface or subsurface storage is required because the proportion of rooftop area is generally too small to hold the required storage.

b) Applicability

Most applicable to new structures with flat rooftops, although existing structures can be used if they meet specific design requirements. Rooftop detention is believed to be most appropriate in urban areas having 50 percent or more low-rise or commercial establishments.

c) Advantages and Disadvantages

ADVANTAGES

- No additional land requirements may be needed.
- Not unsightly or a safety hazard.
- Minimal interference with traffic or people.
- Water stored in rooftop reservoirs has great potential for multiple uses such as grass watering and various washing and cleaning operations.
- May be adaptable to existing structures.

DISADVANTAGES

- The effects of just a few applications are negligible on a watershed basis.
- Benefits to a homeowner may not outweigh the costs.
- May require modifications to building codes before practice can be used.

- o Leaks can cause damage to buildings and their contents.

d) Maintenance Requirements

Routine inspection is desirable to determine how well rooftop detention facilities are meeting their design standards; to check for the possible removal of roof drain control devices (such action may have been taken as a result of leaking roofs); and to determine when cleaning or repairs are needed.

4. Cistern Storage

a) Description

A cistern is a tank or reservoir to which runoff is directed which may be designed as a detention facility with slow release or as a holding tank to store the water for alternative uses.

b) Applicability

Since the function of cisterns is not dependent upon physiographic conditions and their sizes can vary as necessary, they are applicable practically anywhere. Cisterns can be installed beneath paved areas or other structural facilities, within a building, or above the ground.

c) Advantages or Disadvantages

ADVANTAGES

- o Minimal interferences with traffic or people.
- o Can be used in existing as well as newly developed areas.
- o Potential for multiple use of stored runoff may be possible.

DISADVANTAGES

- o Subsurface excavation could be costly depending upon the type and amount of rock encountered.

d) Maintenance Requirements

Periodic removal of sediment and debris will be necessary to assure maximum operating efficiency. If cistern pumps are employed, routine maintenance and inspections will be necessary to minimize failure.

CHAPTER VII. REVIEW OF STORM WATER COLLECTION SYSTEMS AND THEIR IMPACTS

A. Existing Storm Water Collection Systems and Their Impacts

The existing storm water collection and conveyance systems throughout the Monocacy Creek Watershed have been documented through correspondence with the municipalities and field surveys. Much of the existing data was available from work performed for the JPC "Regional Storm Drainage Plan" (RSDP) in the early 1970's. Each municipality was contacted to obtain updated data on the existing storm sewer systems which was added to the RSDP data and mapped on the working base maps of the watershed. For each storm sewer system, the area draining to the system was identified from the topography of the area.

The existing storm water collection and conveyance system was incorporated into the computer model of the watershed as follows:

- o Subareas (which represent the smallest watershed breakdown for modeling purposes) were drawn to be consistent with the areas drained by storm sewers, i.e. the area drained by any one storm sewer system would be wholly within one subarea.

- o Where applicable, major storm water collection/conveyance facilities have been incorporated into the runoff model as "reaches." A reach in the model is a channel segment which forms the link between subareas and establishes the timing relationships between subareas.

Therefore, the existing storm sewer system is part of the documented "baseline" condition for both modeling purposes and for development of the watershed plan.

There are only two man-made storm runoff conveyance facilities used as reaches in the Monocacy Creek hydrologic model. The two reaches are consecutive sections of the open channel along Route 378 between Eaton Avenue and the confluence of the channel with the Monocacy Creek mainstem near Mauch Chunk Road. Located to the north and east of Route 378, the open channel drains nearly four square miles within the City of Bethlehem, Hanover Township, Northampton County and Hanover Township, Lehigh County. The upstream part of the open channel near Eaton Avenue has been designated reach number 96 of the hydrologic model. It is a concrete channel which is fifteen (15) feet wide at the bottom and five (5) feet deep with sides that slope approximately at a ratio of 2 feet horizontal to 1-foot vertical. The downstream part of the open channel has been designated as reach number 97. It is composed of a sediment and rock-bottomed channel immediately downstream of reach 96 and a concrete channel/culvert system beginning at the Eighth Avenue cloverleaf at Route 378. The sediment and rock-bottomed channel is approximately 10 feet wide at the bottom with 10-foot high banks of variable side slope. The concrete channel near Eighth Avenue is 28 feet wide at the bottom with two-foot high concrete sides and with an additional natural embankment ten feet high at minimum. Since only one cross-section may be used to describe a given reach, the rock and sediment-bottomed section was used because it is much longer (i.e. more representative) than the concrete section.

Presented in Table 12 are the location, description and approximate hydraulic capacity of the two open channels used as model reaches. The approximate capacities were calculated using the Manning formula.

TABLE 12

Man-made Storm Water Conveyance Facilities Used as Reaches in the Monocacy Creek Model

	Location	Description	Approximate Capacity
1.	East side of Route 378 at Eaton Avenue Bridge	Trapezoidal concrete channel	2,400 cfs
2.	North side of Route 378 between Eaton Avenue and 8th Avenue	Trapezoidal sediment and rock-bottomed channel	1,800 cfs

Presented in Table 13 is a comparison of the approximate flow capacity of each channel section relative to peak flow values generated by the Penn State Runoff Model for the Monocacy Creek Watershed.

TABLE 13

Man-made Reach Flow Capacity Versus PSRM* Peak Flow Values

Reach No.	Approximate Capacity	PSRM Peak Flow for Return Period: 2-Yr.	10-Yr.	25-Yr.	50-Yr.	100-Yr.
96	2,400 cfs	672	1,110	1,500	1,800	2,300
97	1,800 cfs	840	1,390	1,870	2,230	2,850

*Penn State Runoff Model calibrated for the Monocacy Creek Watershed.

From Table 13, Reach 96 should be capable of transporting peak flows up to and including the 100-year return period event. Reach 97, however, can only convey the peak flow value up to approximately a 25-year return period.

For storm sewer systems which were not part of the model structure, the effectiveness of the systems can only be addressed in the context of whether they coincide with documented storm drainage problem areas. No flow data is available from the modeling process to identify capacity deficiencies for "non-reach" storm sewer facilities. A review of the storm drainage problem area inventory versus the storm sewer inventory indicates that only six of the thirty-eight storm drainage problem areas are in areas served by non-reach storm sewers. This data indicates that in the large majority of instances, the existing storm water collection facilities are effective in minimizing the "local" storm runoff impact of development.

B. Future Storm Water Collection Systems

As part of the process of documenting the existing storm water collection network throughout the watershed, an attempt was made to identify proposed drainage facilities also. In general, data regarding proposed facilities is very sketchy. Typically, storm drainage improvements would be constructed either as part of land developments (by the developer) or as remedial measures as part of the municipal capital or maintenance programs on an as-needed basis. As-needed refers to both the severity of the drainage problem and the public support (or outcry) for an improvement. In this manner, projects are constructed as money becomes available in the capital or maintenance budget. The effect of the approach in most cases is a piecemeal process of storm drainage improvements rather than one based on a comprehensive program keyed to future needs.

The Monocacy Creek Storm Water Management Plan can impact this situation in three ways. First, implementation of the performance standards specified in Chapter VIII would prevent the formation of new storm drainage problems or the aggravation of existing problems by maintaining peak flow values throughout the watershed to existing levels. This would allow for the development of a comprehensive remedial strategy based on the assurance that solutions would not eventually be obsolete with additional development. Second, the storm drainage problem area inventory in Chapter V provides an excellent basis for development of a storm drainage capital improvements inventory. Actual improvements required would be determined from engineering analysis of the problems. Table 10 lists proposed solutions to the existing problems where available. Third, any engineering studies conducted for correcting problem areas could benefit from the flow values generated from the computer modeling of the watershed as part of this plan.

Even without the development of a comprehensive remedial strategy, the Storm Water Management Plan will improve the current situation by specifying a consistent design philosophy for all future storm drainage facilities. This design philosophy will relate to both facilities associated with new development and remedial projects.

C. Existing and Proposed Flood Control Projects

There is only one existing flood control project impacting the Monocacy Creek Watershed. The Corps of Engineers constructed flood walls, a levee and a dike to protect the old industrial area of the City of Bethlehem (within the Monocacy Creek watershed) from the backwater impacts of the Lehigh River flows. The project was completed in 1964 at a total cost of $14.2 million. There are no other existing or proposed flood control projects within the Monocacy Creek Watershed.

CHAPTER VIII. WATERSHED-LEVEL RUNOFF CONTROL PHILOSOPHY AND PERFORMANCE STANDARDS FOR THE CONTROL OF STORM WATER RUNOFF FROM NEW DEVELOPMENT

Earlier chapters identified the impacts of new development on storm water runoff and the techniques available to control those impacts either on-site or with regional facilities. This chapter will identify the performance standards or goals which need to be met for various areas of the watershed to minimize the adverse storm water impacts of new development. The method used to determine the performance standards was the development of a detailed hydrologic computer model of the watershed which could be "stressed" under various design conditions to evaluate control options. The specific computer model used was the Penn State Runoff Model (PSRM) because it provides acceptable hydraulic and hydrologic accuracy, has minimal input data requirements, produces total runoff data and not merely peaks, and can be programmed on a micro-computer. An additional advantage was the subsequent decision to use two Pennsylvania State University professors as engineering consultants for the watershed plan - one of whom is the principal author of PSRM.

Construction of the computer model of the Monocacy Creek Watershed first involved breaking the watershed down into small pieces of approximately 300 acres average size. These pieces, or subareas are the building blocks of the model. For each of the 101 subareas, the computer model generates a runoff hydrograph (flow versus time) for a particular rainfall event. Stream channel data provides the linkage between subareas and establishes the timing of one part of the watershed relative to another. The model provides the tool for analysis of the watershed and determination of an appropriate control strategy. The manner in which the model has been used to develop the control strategy and the actual control strategy itself are discussed in the following sections.

A. Watershed-Level Runoff Control Philosophy

Historically, storm water management decisions for new development in the Monocacy Creek Watershed have predominantly been made using "at-site" philosophy. This has been the case for two reasons. First, four out of eleven municipalities in the watershed do not require consideration of the downstream impacts of storm runoff from new developments in their subdivision ordinances. Second, for those municipalities which attempt to consider downstream impacts, the municipal engineers do not have a watershed data-base to rely on to quantify those impacts. The bottom line, therefore, is that at-site considerations would typically dictate the recommended controls.

The difference between at-site runoff control philosophy and the Act 167 watershed-level philosophy is the consideration of downstream impacts. Whereas the objective of typical at-site design would only be to control post-development peak runoff rates to pre-development levels from the site itself, a watershed-level design would be geared towards maintaining existing peak flow rates in the entire drainage system. The latter requires knowledge of how the site relates to the entire watershed in terms of the timing of peak flows, contribution to peak flows at various downstream locations and the impact of the additional runoff volume generated by development of the site. The proposed watershed-level runoff control philosophy is based on the assumption that runoff volumes <u>will</u> increase with development and, rather than necessarily attempting to reduce post-development volume seeks to "manage" the increase in volumes such that peak rates of runoff throughout the watershed are not increased.

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

Presented in Figure 11 is a typical at-site runoff control strategy for dealing with the increase in the peak rate of runoff with development. The "Existing Condition" curve represents the predevelopment runoff hydrograph. The "Developed Condition" hydrograph portrays three important changes in the site runoff response with development - a higher peak rate, a faster occurring peak (shorter time for the peak rate to occur), and an increase in total runoff volume. The "Controlled Developed Condition" hydrograph is based on limiting the post-development runoff peak rate to the predevelopment level through use of detention facilities. The impact of "squashing" the post-development runoff to the predevelopment peak is that the peak rate occurs over a much longer period of time. The instantaneous predevelopment peak has become an extended peak (approximately two hours long in this example) under the Controlled Developed Condition.

At-site, the maintenance of the predevelopment peak rate of runoff is an effective management approach. The potential detrimental impact of this approach is illustrated by Figures 12 and 13. Figure 12 represents the existing hydrograph at the point of confluence of Watershed A and Watershed B. The timing relationship of the watersheds is that Watershed A peaks more quickly (at time t_{pA}) while Watershed B peaks more slowly (at time t_{pB}), resulting in a combined time to peak approximately in the middle (at time T_p). Watershed A is an area of significant development pressure and all new development proposals are met with the at-site runoff control philosophy as depicted in Figure 11. The eventual end

FIGURE 11
TYPICAL "AT-SITE" RUNOFF CONTROL PHILOSOPHY

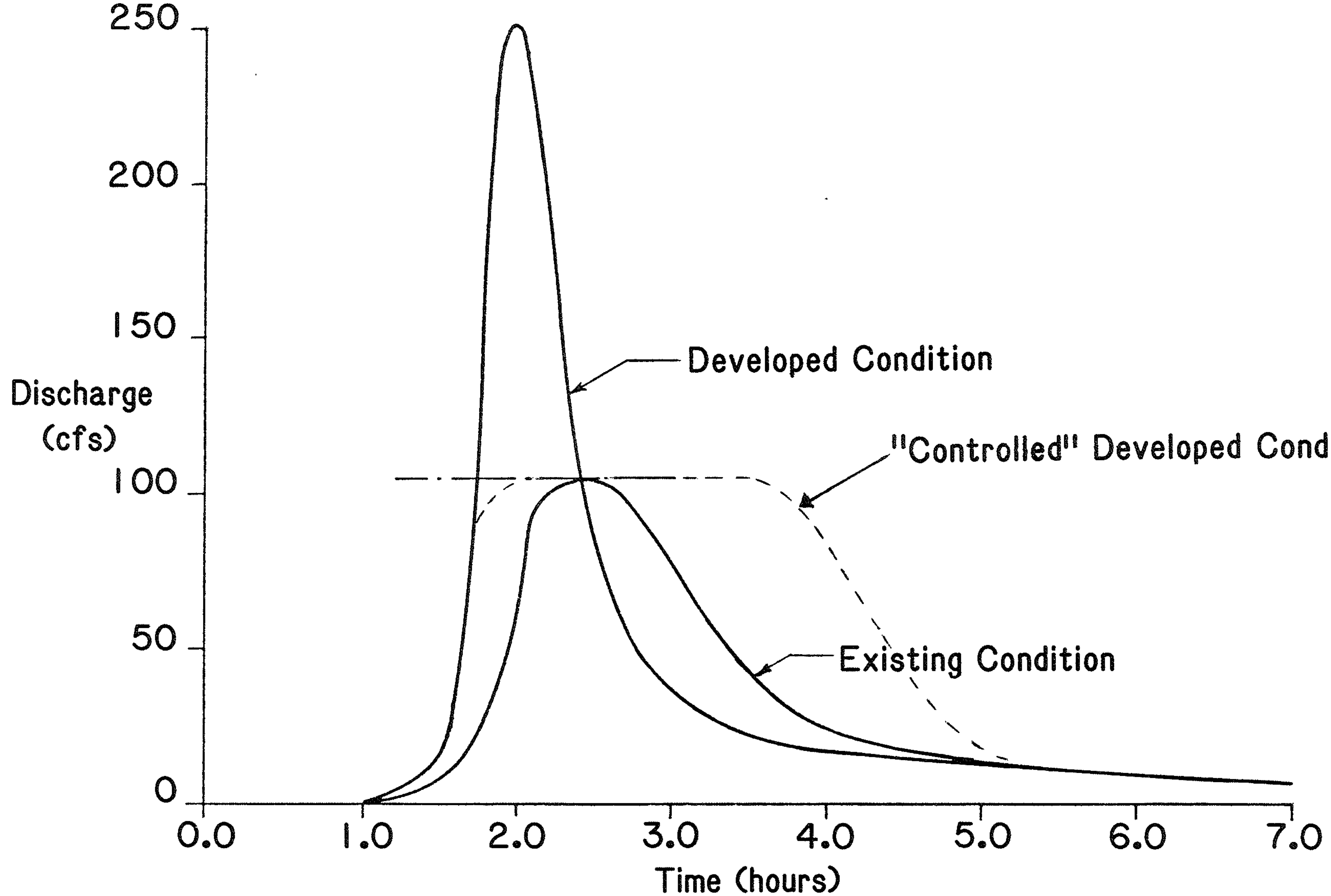


FIGURE 12
EXISTING RUNOFF CONDITION
Predevelopment

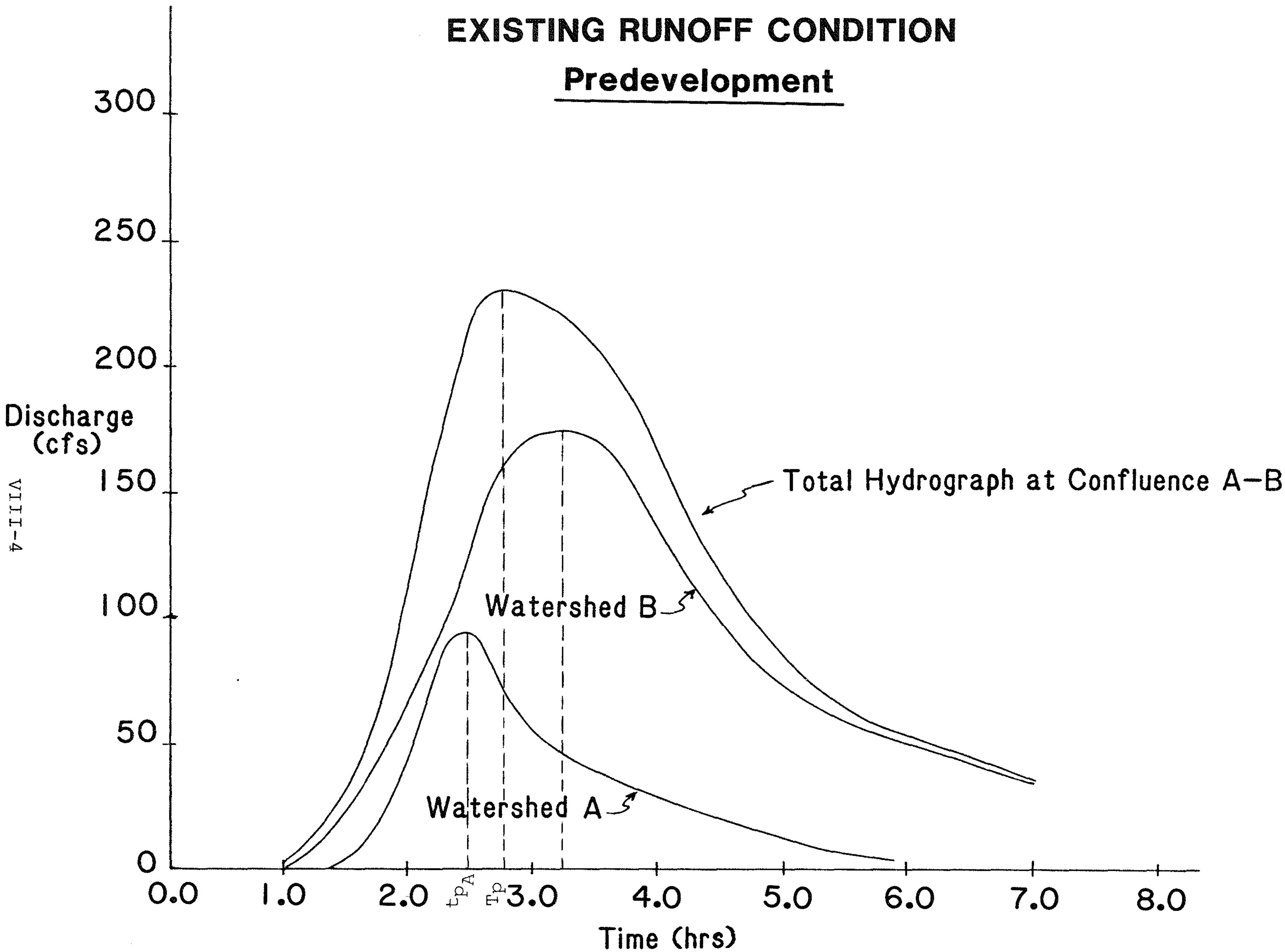
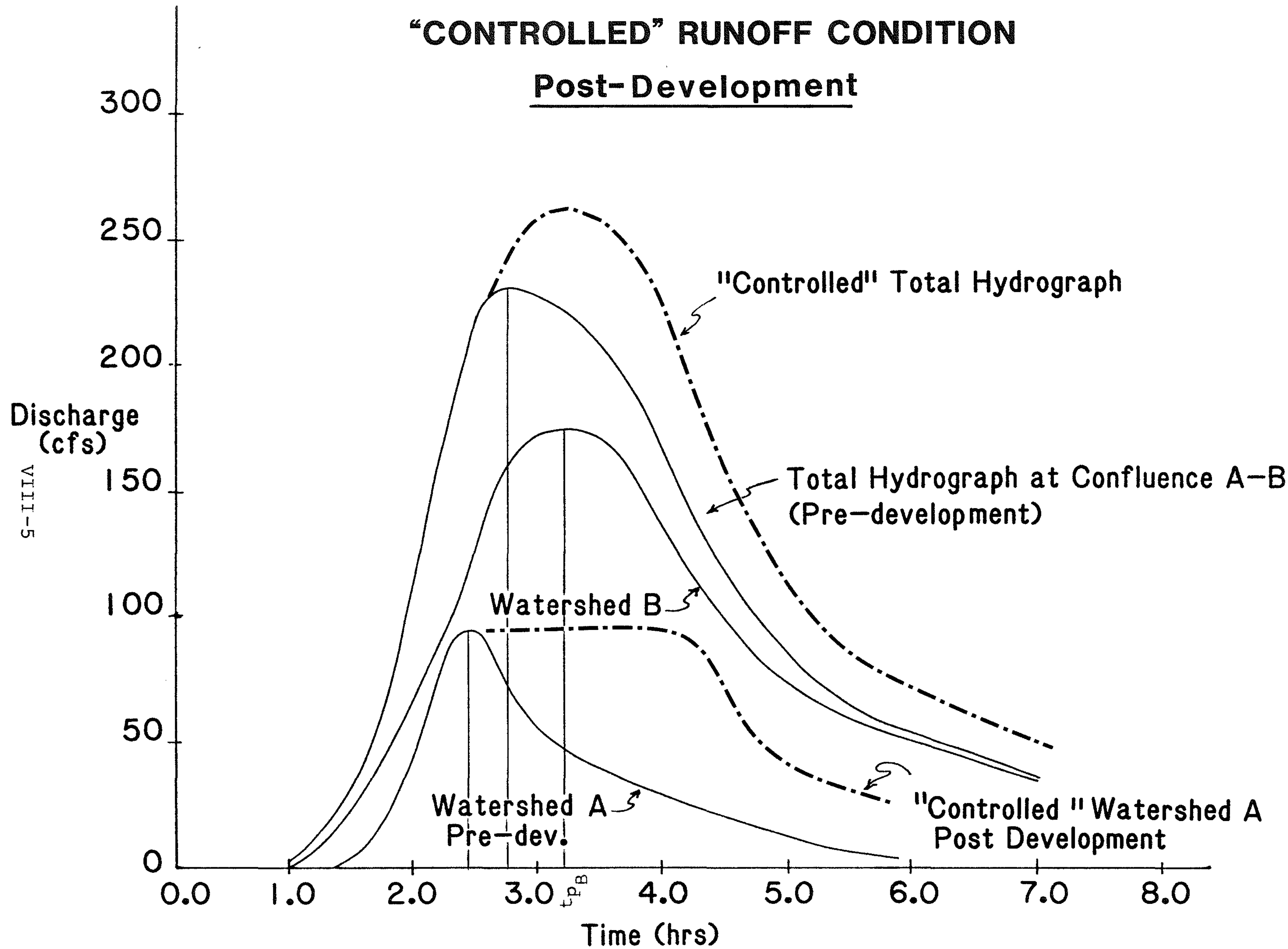


FIGURE 13
"CONTROLLED" RUNOFF CONDITION
Post-Development



product of the Watershed A development under the "Controlled" Runoff Condition is an extended peak rate of runoff as shown in Figure 13. The extended Watershed A peak occurs long enough so that it coincides with the peak of Watershed B. Since the total hydrograph at the confluence is the sum of A and B, the total hydrograph peak must increase under these conditions to the "Controlled" Total Hydrograph. The conclusion from the above example is that simply controlling peak rates of runoff at-site does not guarantee an effective watershed-level control because of the increase in total runoff volume.

1. Release Rate Concept

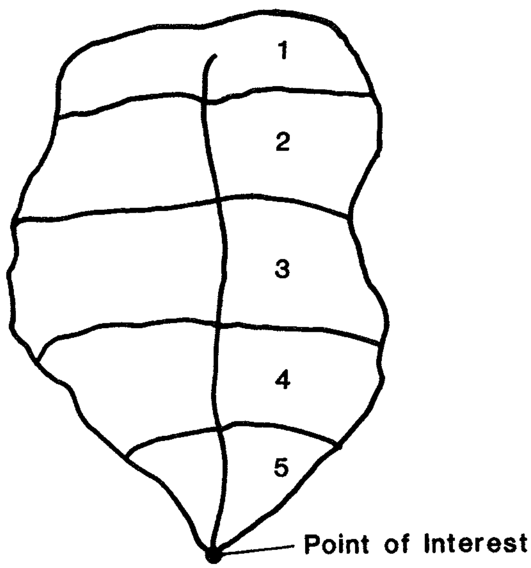
The previous example indicated that in certain circumstances it is not quite enough to control post-development runoff peaks to predevelopment levels if the overall goal is no increase in peak runoff at any point in the watershed. The reasons for this are how the various parts of the watershed interact, in time, with one another and the increased volume of runoff with development. The critical runoff control criteria for a given site or watershed area is not necessarily its own predevelopment peak rate of runoff but rather the predevelopment contribution of the site or watershed area to the peak flow at a given point of interest. This concept is best explained through the use of a few simplified charts.

Figure 14 indicates how the individual runoff contributions from a number of sites or watersheds create the total hydrograph at a particular point. Areas 1 through 5 each have a particular runoff response to a given rainfall event (i.e. each will generate a characteristic hydrograph). Note that the configuration of the watershed is such that all areas will contribute runoff to the point of interest at the downstream end of area 5. The five areas do not contribute at the same time, however. Flows from area 1 have the furthest to go to get to the point of interest. Area 5 flows contribute immediately to the point of interest flows. The contribution of each area to the hydrograph at the point of interest, therefore, is the individual area hydrograph lagged in time by an amount equal to the travel time from the area to the point of interest. The total hydrograph at the point of interest and the individual contributions from areas 1 through 5 are shown in Figure 14.

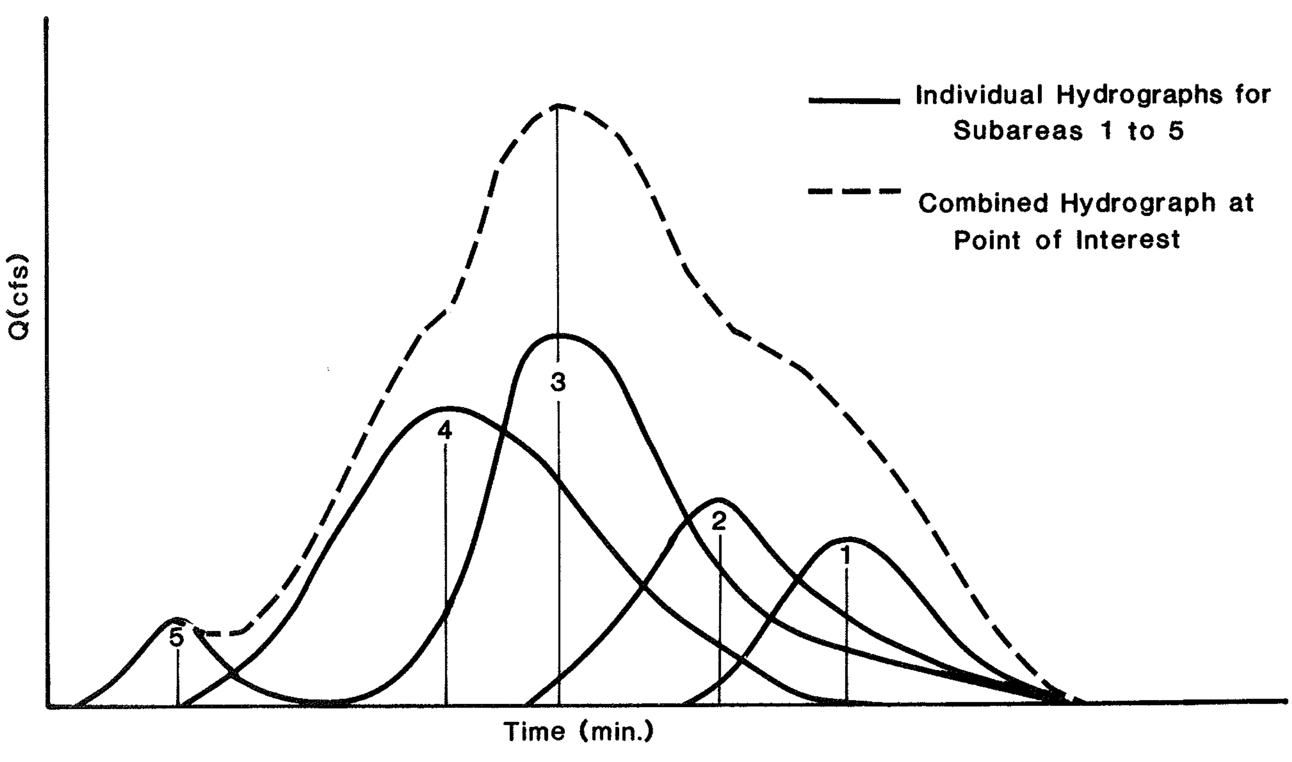
The release rate concept is perhaps best described by looking at how area 4 contributes to the hydrograph at the point of interest. Figure 15 shows the total

FIGURE 14
"POINT OF INTEREST" HYDROGRAPH ANALYSIS EXAMPLE

Watershed Configuration



Hydrograph Components at Point of Interest



hydrograph from Figure 14 and the area 4 contribution only. Noteworthy facts regarding the two hydrographs are that area 4 itself peaks before the peak of the total hydrograph (40 minutes versus 50 minutes), the peak flow from area 4 is 100 cfs and the contribution of area 4 to the peak flow at the point of interest is 75 cfs. Also shown on Figure 15 are the possible outcomes of development occurring in area 4. Specifically, the possible area 4 hydrograph assuming development occurs with no storm water controls and the resultant hydrograph if all new development uses the at-site philosophy of controlling to predevelopment peak levels are shown. Note that in both cases the flow contribution of area 4 to the peak at the point of interest increases (85 cfs for the "no control" option and 100 cfs for the "at-site" philosophy option). Obviously, therefore, the total peak flow at the point of interest from areas 1 through 5 must increase for both options and neither is an acceptable control strategy. The only acceptable control strategy would be to ensure that the contribution of area 4 to the peak flow at the point of interest does not exceed 75 cfs. Note that the 75 cfs represents 75% of the 100 cfs peak flow from area 4. Herein lies the basis for the release rate concept.

Conventional at-site detention philosophy would control post-development peak runoff flows to <u>100%</u> of predevelopment levels. The release rate concept would dictate a more stringent level of control based on downstream conditions. For area 4, the release rate would be 75% meaning that each individual development within area 4 would have to control post-development peak runoff rates to 75% of predevelopment levels as illustrated in Figure 16. Only through this increased level of control for area 4 would the point of interest peak flows not be exceeded. The conclusion, therefore, is that in exchange for increased runoff volume with development the peak rate of runoff will actually need to be reduced relative to predevelopment conditions for certain parts of the watershed. The release rate for those watershed areas, or subareas, is defined in equation form as follows:

$$\text{Release Rate} = \frac{\text{Subarea Contribution to Point of Interest Peak}}{\text{Subarea Peak Flow}}$$

Note that the release rate concept has been developed using area 4 from Figure 14 as an example. The characteristics of area 4 are that it peaks prior to the point of interest peak <u>and</u> it contributes flow to the point of interest peak flow. None of the other areas in

FIGURE 15
HYDROGRAPH ANALYSIS FOR EXAMPLE SUBAREA 4

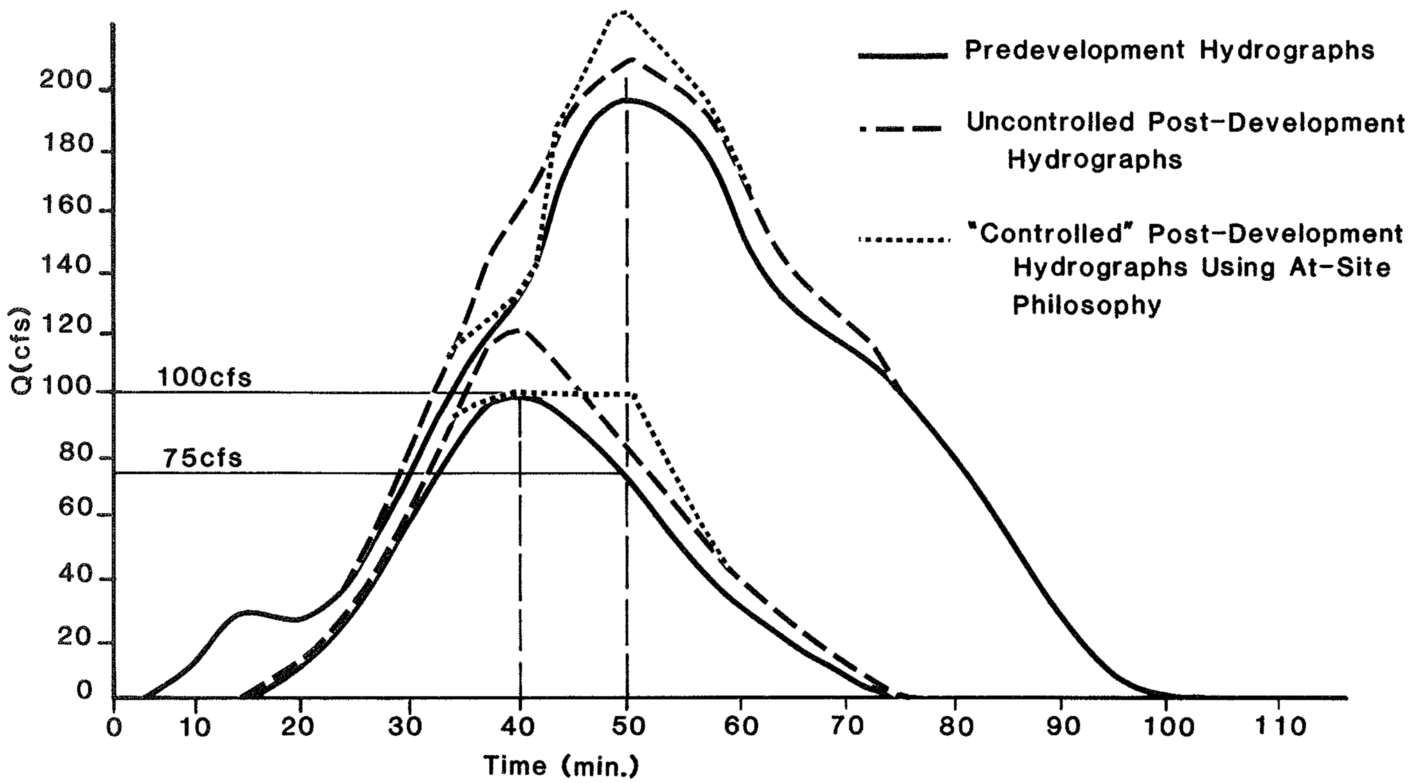
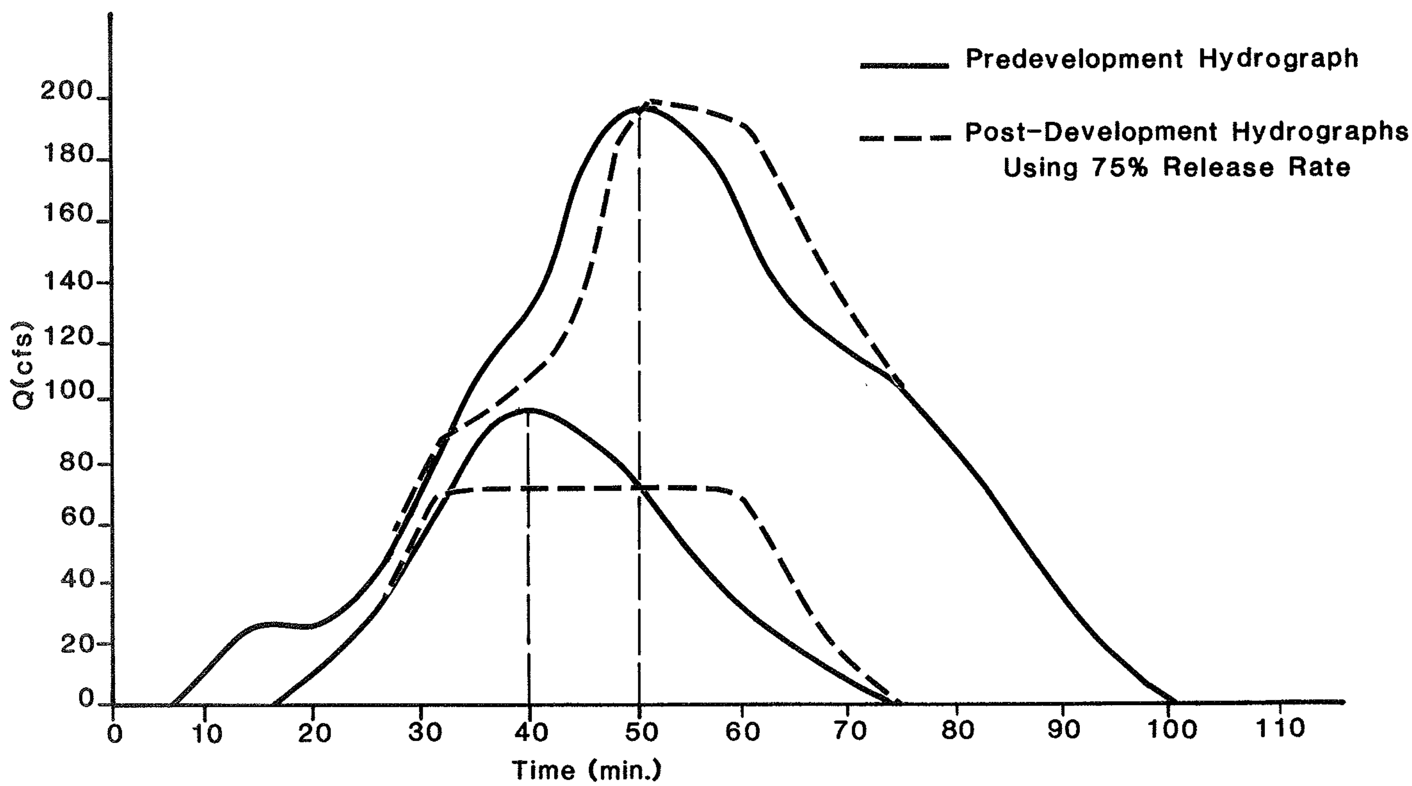


FIGURE 16
RELEASE RATE RUNOFF CONTROL FOR EXAMPLE SUBAREA 4



the example (1, 2, 3 or 5) exhibit both of these characteristics. As such, the proper method of runoff control applicable to these areas may differ from the basic release rate control strategy as discussed in the following section.

2. Runoff Control Strategy Categorization

The five drainage areas of the previous example beginning with Figure 14 each contribute to the runoff at the point of interest in a different manner as outlined below:

Area 1: Due to its very long travel time, area 1 peaks later than the point of interest peak and does not contribute any runoff to the point of interest peak.

Area 2: Due to its long travel time, area 2 peaks later than the point of interest peak but does contribute to the point of interest peak.

Area 3: Area 3 peaks at exactly the same time as the point of interest peak due to its location in the middle of the watershed. Therefore, 100% of the area 3 peak contributes to the point of interest peak.

Area 4: Area 4 peaks prior to the point of interest peak and contributes to the point of interest peak.

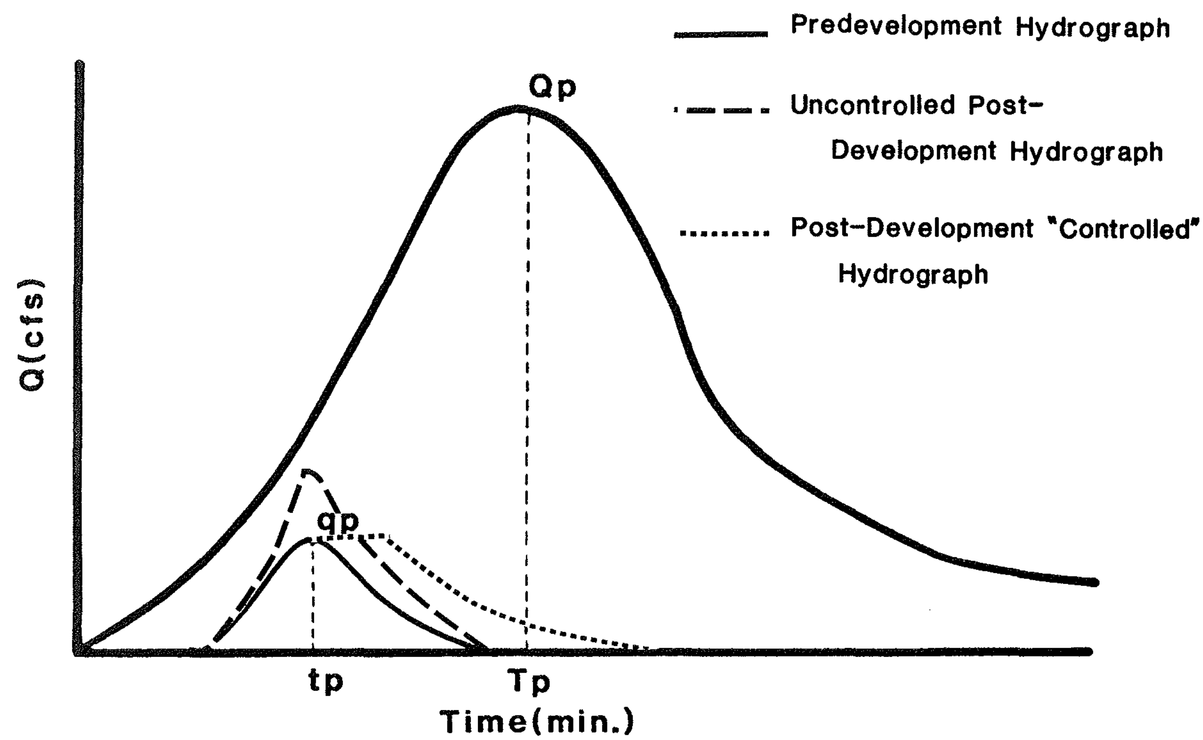
Area 5: Due to its proximity to the point of interest, area 5 peaks very early (before the point of interest peak) and does not contribute to the point of interest peak.

Each of the above situations presents a different storm runoff analysis problem and, in fact, the five areas define the five different runoff categories which need to be examined in the preparation of a watershed level runoff control plan. The five categories, or cases, are described in the sections below.

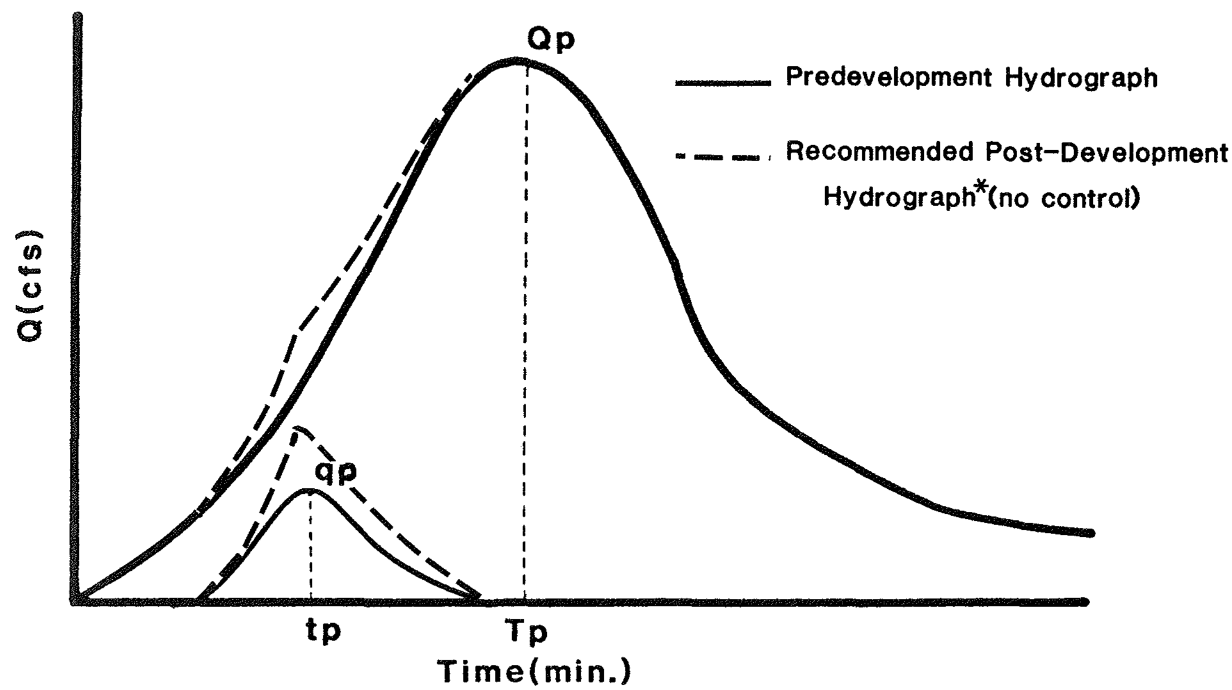
(a) Case I (Equivalent to Area 5) - Figure 17 portrays the Case I example of a drainage area which peaks prior to the point of interest peak and does not contribute to the peak flow of interest. From Figure 17, q_p and t_p are the peak flow and time to peak, respectively, of the individual drainage area and Qp and Tp are the peak flow and time to peak, respectively, of the hydrograph at the point of interest. In addition, the value of the

FIGURE 17
CASE I ANALYSIS CATEGORY AND RUNOFF CONTROL STRATEGY

CASE I: $t_p < T_p$ <u>and</u> subarea does not contribute to subwatershed peak



Recommended Watershed-Level Runoff Control Strategy



*Contingent upon ability of localized drainage network to safely convey higher peak runoff.

individual drainage area hydrograph at any point in time is specified as q @ t, where t is the time in question (e.g. q @ 0 = 0, q @ t_p = q_p , q @ T_p = 0). Therefore, notationally, Case I is described as follows:

$t_p < T_p$ and q @ $T_p = 0$ ("<" means less than)

Application of the basic release rate concept to Case I would dictate a release rate of 0% corresponding to the contribution of the drainage area to the point of interest peak. Taken literally, a 0% release rate would mean no water would leave the site post-development. Obviously, this would not be a workable control and, in fact, not a necessary one. The reason is that a release rate does not have to be associated with a detention facility geared to pass a certain percentage of predevelopment peak flows. The release rate applicable to Case I is that whatever the storm runoff control philosophy used, the contribution of the individual drainage area to the point of interest peak should be zero. The most appropriate control in this instance is no control as shown in Figure 17. Whereas any form of detention may extend the peak flow such that the drainage area begins to contribute to the point of interest peak, simply allowing the drainage area hydrograph to peak higher and recede in an uncontrolled fashion results in a more effective approach at the point of interest. Note that the impact of the no control approach for the subarea on the point of interest hydrograph is limited to the rising limb of the hydrograph and not the peak. The Case I runoff control philosophy, therefore, would be no control at all provided that the unrestricted runoff can be safely transported to the stream channel from each development site.

(b) Case II (Equivalent to Area 4) - Figure 18 portrays the Case II example of an area which peaks prior to the peak at the point of interest and does contribute to the peak or, notationally:

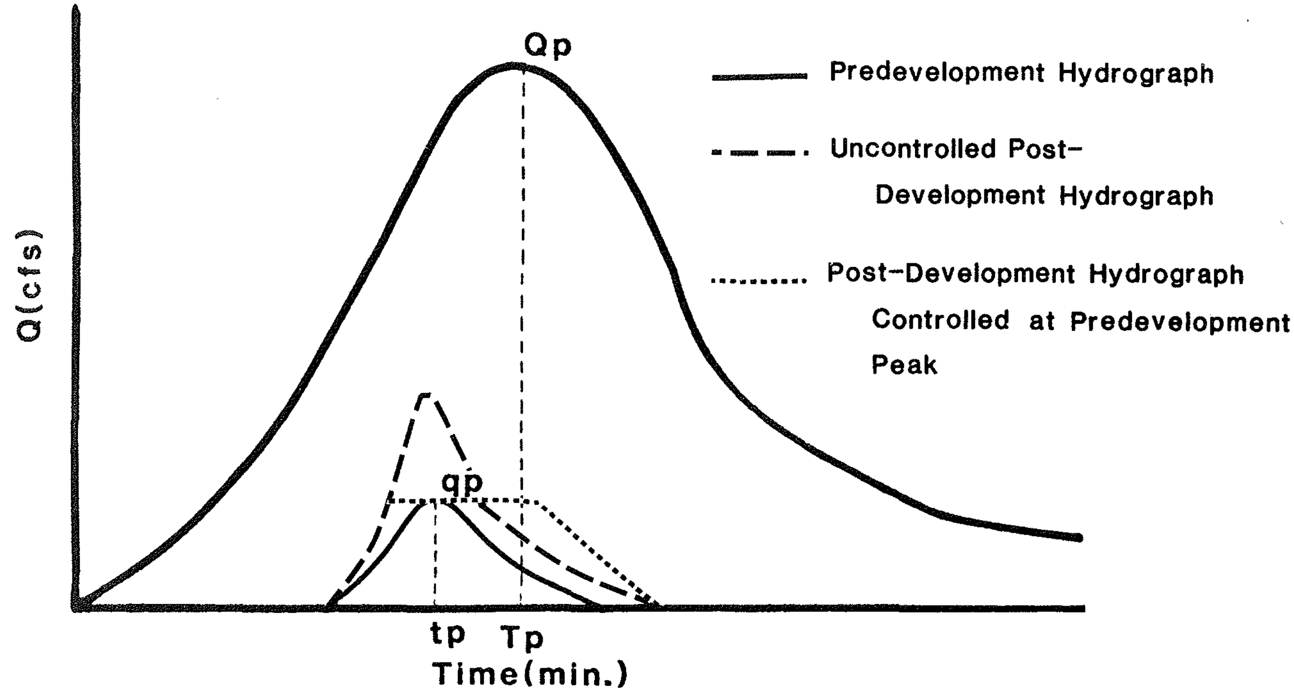
$t_p < T_p$ and q @ $T_p > 0$ (">" means greater than)

The calculated release rate for this situation could fall anywhere within the range of 1% to 99% depending upon the difference between t_p and T_p for various drainage areas which contribute to the point of interest. A 99% release rate area represents essentially the conventional (Case III)

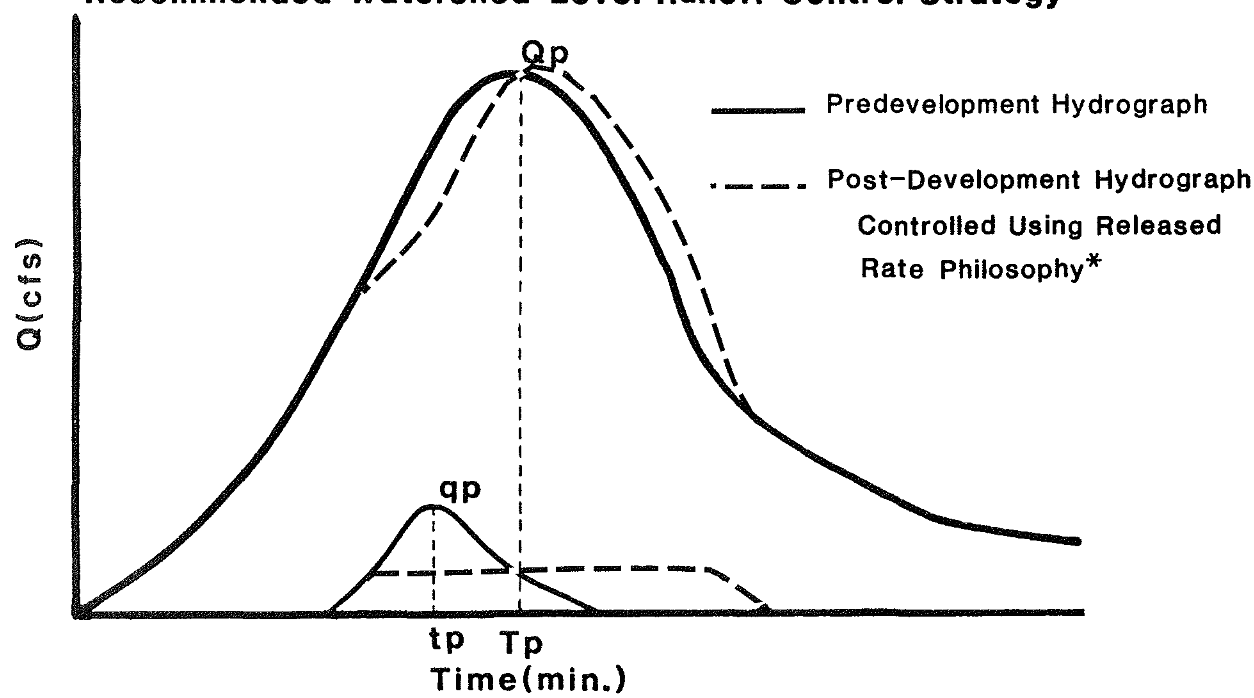
FIGURE 18

CASE II ANALYSIS CATEGORY AND RUNOFF CONTROL STRATEGY

CASE II: $t_p < T_p$ <u>and</u> subarea does contribute to subwatershed peak



Recommended Watershed-Level Runoff Control Strategy



*For watershed areas which fall into Case II and have very low Release Rates the Case I option would be employed. See text for additional details.

detention philosophy of controlling to the predevelopment peak level. The 1% release rate area essentially is a Case I area where, rather than attempting to detain the runoff from new development to 1% of predevelopment levels, a no control approach would be adopted. Within the range of 1% to 99%, however, the appropriate control strategy is not always so clear as will be discussed in Chapter VIII - Section A.4..

(c) Case III (Equivalent to Area 3) - The Case III situation is presented in Figure 19. Case III represents the simplest control strategy where the release rate is 100% since the time to peak of the drainage area equals the time to peak of the point of interest. For Case III areas, detention should be provided to ensure that post-development peak runoff does not exceed predevelopment levels.

(d) Case IV (Equivalent to Area 2) - Figure 20 displays the Case IV situation where the individual drainage area peaks later than the point of interest peak and the drainage area contributes to the point of interest peak. Again, notationally:

$$t_p > T_p \text{ and } q @ T_p = 0$$

Case IV does not fit the conventional release rate concept because of the relationship between the times to peak. However, as depicted on Figure 20, uncontrolled post-development runoff could increase the point of interest peak because of the tendency of new development to raise the peak of the drainage area <u>and</u> decrease the time to peak. The appropriate control strategy would be to simply provide detention for the drainage area designed to slow the rise of the hydrograph to the predevelopment level and control peak flows to the predevelopment condition.

(e) Case V (Equivalent to Area 1) - The Case V situation is shown in Figure 21 where the drainage area time to peak occurs much later than the point of interest peak and the drainage area does not contribute to the point of interest peak, or:

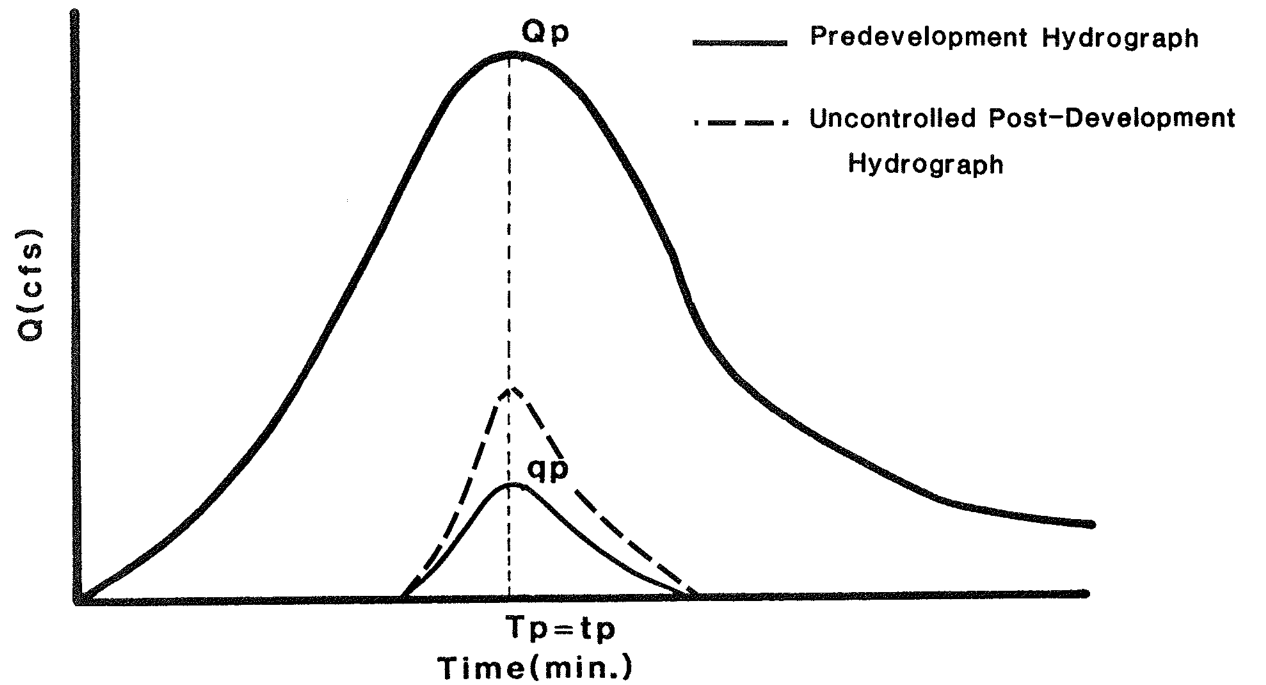
$$t_p > T_p \text{ and } q @ T_p = 0$$

The runoff control strategy adopted for Case V areas is very nearly inconsequential at the point of interest. Neither uncontrolled post-development runoff nor extended detention-achieved peaks would

FIGURE 19

CASE III ANALYSIS CATEGORY AND RUNOFF CONTROL STRATEGY

CASE III: $t_p = T_p$



Recommended Watershed-Level Runoff Control Strategy

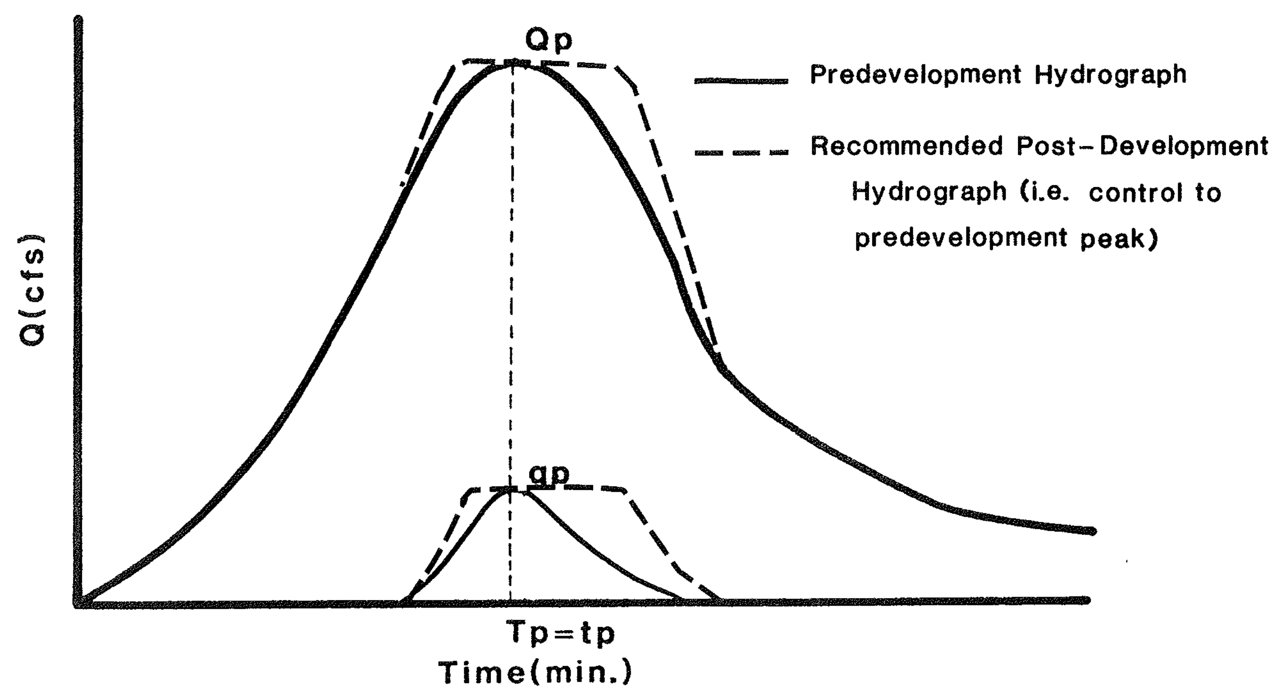
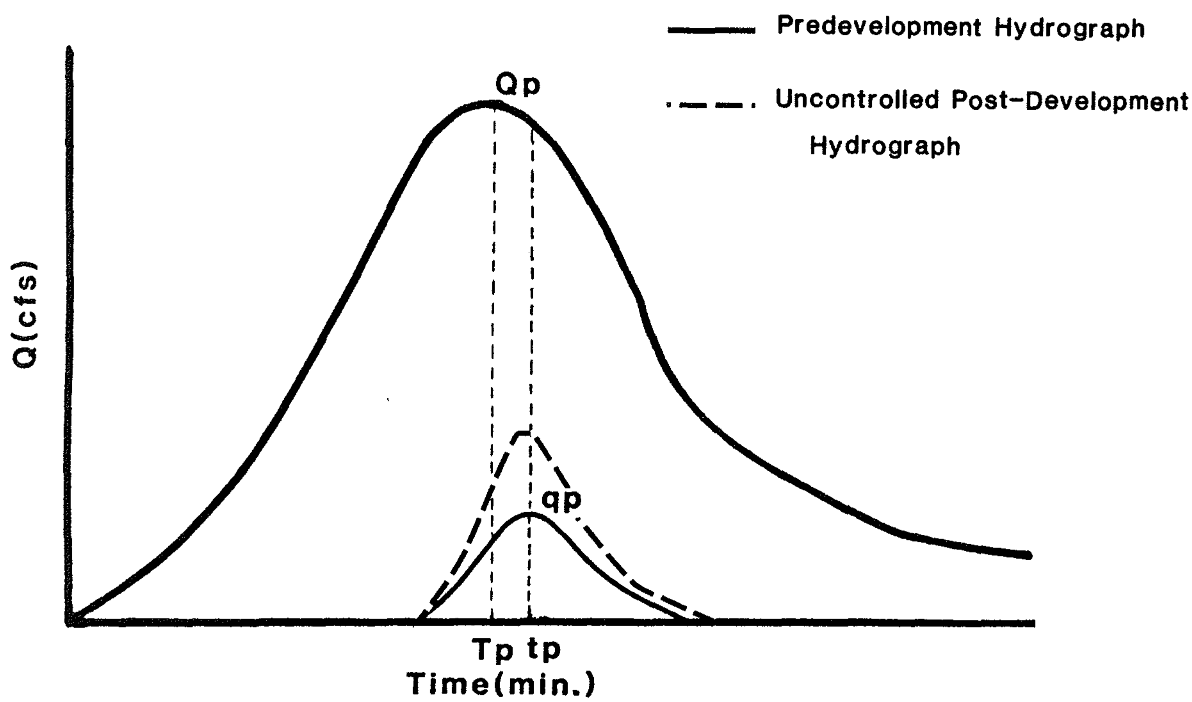


FIGURE 20

CASE IV ANALYSIS CATEGORY AND RUNOFF CONTROL STRATEGY

CASE IV: $t_p > T_p$ <u>and</u> subarea does contribute to subwatershed peak



Recommended Watershed-Level Runoff Control Strategy

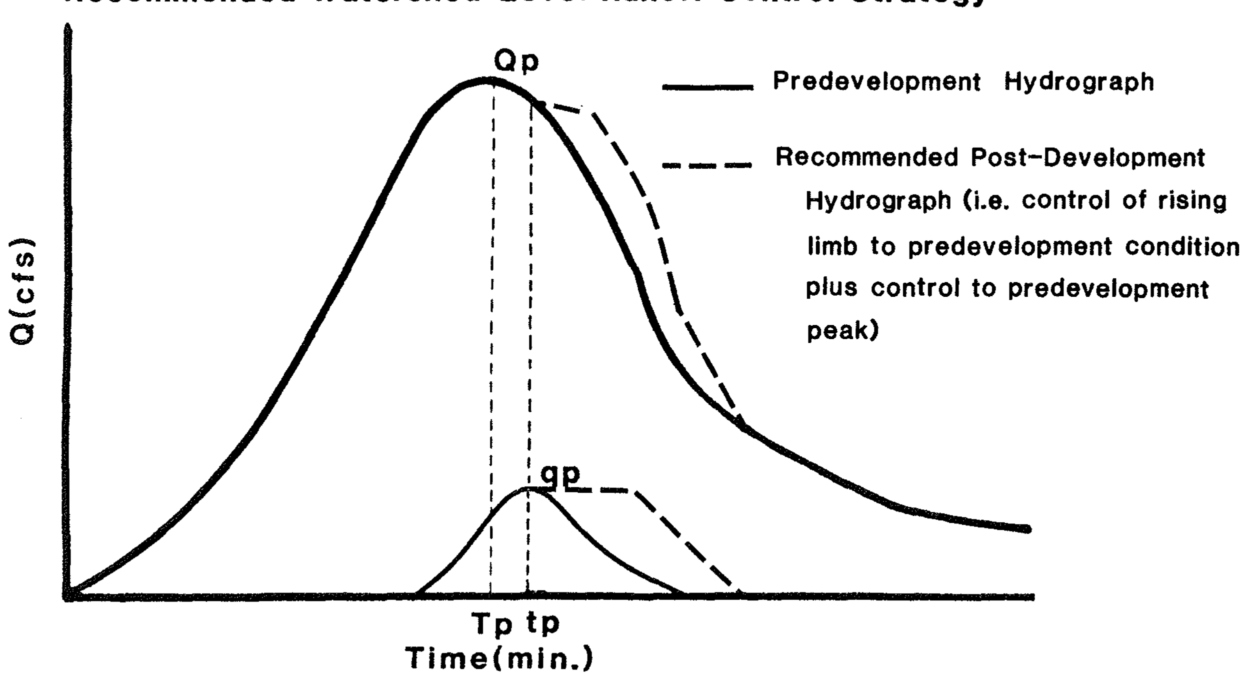
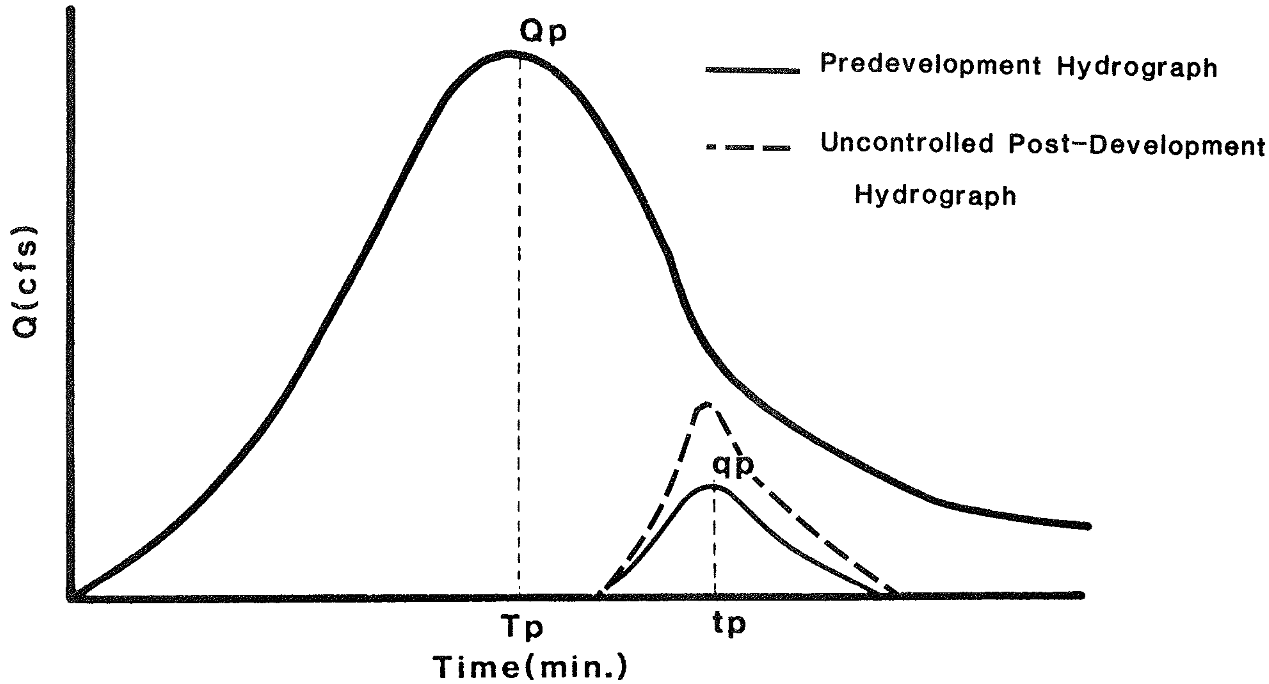
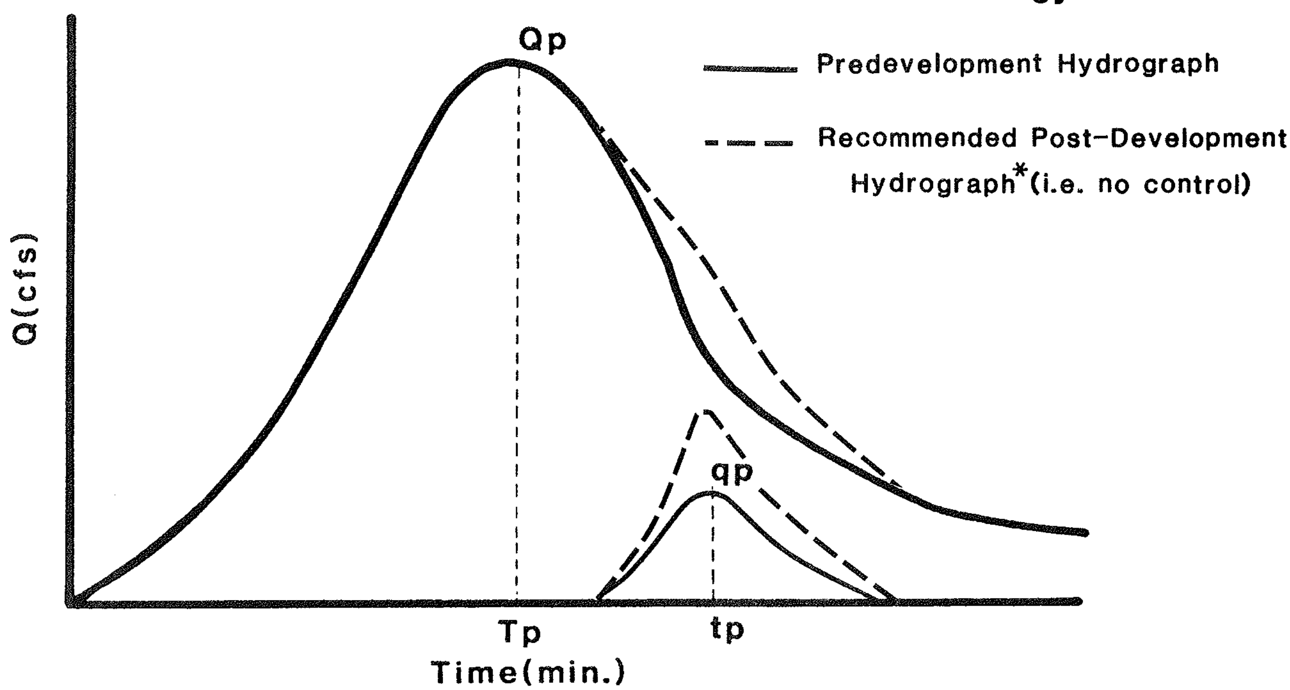


FIGURE 21
CASE V ANALYSIS CATEGORY AND RUNOFF CONTROL STRATEGY

CASE V: $t_p > T_p$ <u>and</u> subarea does not contribute to subwatershed peak



Recommended Watershed-Level Runoff Control Strategy



*Contingent upon ability of localized drainage network to safely convey higher peak runoff.

have the effect of increasing the point of interest peak flow. The control strategy to be adopted, therefore, in the interest of cost-effectiveness, is the no control approach provided that the increased peak can be safely transported to the main runoff channel.

3. Point of Interest Selection

The five runoff control categories, Cases I through V, developed above were determined based on a single point of interest at the downstream end of area 5. This was done simply for ease of illustration. In actuality, however, a point of interest could occur at any location within the sample watershed such as the downstream end of area 1, 2, 3 or 4. Given that the relationships between the point of interest hydrograph and a single drainage area hydrograph (as defined by Cases I - V) are determined by travel time between the drainage area and point of interest, selection of the point of interest would have a definite bearing on the runoff control category each drainage area fits into. Further, selection of multiple points of interest could mean that each drainage area would fit into multiple control categories. Therefore, selection of the points of interest is a critical element in the development of the watershed-level runoff control strategy. For the Monocacy Creek Act 167 Plan, the following items have been considered in the selection of the points of interest:

(a) Existing storm drainage problem areas (38) - identified through the Watershed Advisory Committee municipal representatives

(b) Significant obstructions (16) - identified from detailed Flood Insurance Studies as obstructions which raise flood heights.

(c) All subarea boundaries (101) - identified by breakdown of the watershed for modeling purposes

(d) Municipal boundaries

The overall goals of Act 167 are to prevent the aggravation of existing drainage problem areas and to prevent the formation of new problem areas through the coordination of storm runoff decisions throughout the watershed. At minimum, therefore, existing storm drainage problem areas must be used as points of interest for hydrograph analysis. Of the 38 identified problem areas, 17 are located on main reaches of the runoff model and 21 are located within individual drainage areas. Only the 17 main reach problem areas can be analyzed using the model directly. The remaining 21 problem areas would require a more localized analysis of the impact of potential new development sites which drain through these "off-line" problem areas.

Prevention of any new storm drainage problem areas is by far the more difficult Act 167 goal. Ensuring that no new problems are created requires that either (1) peak runoff values are not increased at any point in the watershed, or (2) peak flow values are only increased to the point that the existing drainage system can safely convey the increased flows. Option 2 would require knowledge of the capacity of the drainage system at every point in the watershed. Certainly this is not the case for the 49.3 square mile Monocacy Creek Watershed. For modeling purposes, the average capacities of the major drainage elements have been determined using simplified methods. Actual capacities may differ significantly depending upon the accuracy of the assumptions used in the simplified approach. In addition, even calibration of the runoff model does not guarantee accurate runoff values at every point in the watershed. The conclusion is that even though it may be possible to increase peak flow values at various points in the watershed without creating new drainage problems, the ability to accurately define those areas and identify the allowable increase in peak flow does not exist within the Act 167 planning effort. Therefore, a conservative engineering approach and practicality dictate using the philosophy of maintaining existing peak flow rates.

With the control philosophy decided, it is still necessary to determine at what points in the watershed the philosophy will be applied. Strict adherence to the philosophy would mean using the most detailed level of watershed breakdown available as the control points, i.e. the 101 subarea boundaries. Using the 101 subarea boundaries as control points would effectively control all of the other possible control options also (i.e. significant obstructions and municipal boundaries as well as the existing main reach problem areas).

Justification for use of significant obstructions as control points would be that ponding currently occurs at these locations indicating a lack of adequate conveyance capacity under existing conditions. Increased peak flows at these points would aggravate the current ponding conditions and possibly create a hazard to property or safety.

Municipal boundaries as possible control points have their justification in the goals of Act 167 itself, namely to coordinate the runoff control efforts of all the municipalities in the watershed. Municipal coordination could mean, at minimum, that the storm water management decisions made for a development in one municipality do not have an adverse impact on any other downstream municipality. Therefore, using municipal boundaries as points of interest could ensure the minimum acceptable coordination consistent with Act 167.

Each of the individual control point categories (existing drainage problem areas, significant obstructions and municipal boundaries) are valid control points for formulation of a runoff management plan. Since, as stated above, using the 101 subarea boundaries effectively incorporates all the other control categories, the 101 subarea boundaries have been used as the critical drainage points for runoff analysis. Therefore, the runoff from a particular subarea has been analyzed at every other downstream subarea and the appropriate control philosophy devised based on not increasing the peak flow at any of the 101 subarea boundaries.

Devising a runoff control strategy based upon 101 critical points means that each subarea in the watershed will fit into multiple control strategy categories (Cases I through V). The control strategy selected for a particular subarea is based on the most critical category applicable to the subarea. One impact of this is that there are no subareas for which the Case V situation is most critical since evaluation of upland-most subareas at their own downstream points yields a 100% release rate. Further, only in very isolated instances would a Case IV situation be most critical. Therefore, the control strategy developed is based essentially on runoff control categories I through III.

4. Minimum Reasonable Release Rate Determination

Application of the control point philosophy of using all 101 subarea boundaries results in release rates throughout the watershed varying from 1% to 100% as generated by the Penn State Runoff Model (PSRM). A 1% release rate would apply to a subarea located near the mouth of the creek which peaks very early with respect to the point of interest peak and only contributes 1% of its own peak to the point of interest peak. A 100% release rate would apply to a subarea near the headwaters of the creek which peaks at the same time as the point of interest peak. Specification of a 100%

release rate as a performance standard would represent the conventional approach to runoff control from at-site philosophy, namely, controlling the post-development peak runoff to predevelopment levels. This is a well-established and technically feasible control which is effective at-site and, where appropriate, will be an effective watershed-level control. Conversely, specification of a 1% release rate as a performance standard would not be feasible from either a technical or cost standpoint for most developments. Controlling post-development peak flows to 1% of predevelopment levels would in most instances require extraordinarily large and expensive detention basins which would only be slightly more effective than a "no detention" approach from a watershed perspective as described in Section A.2. above. Therefore, the preferred approach for a 1% release rate area would be the no detention option (local conveyance conditions permitting).

For areas with release rates between 1% and 100%, the most appropriate runoff control option is not always so clear. As release rates decline from 100%, the technical and/or cost feasibility to achieve the release rate diminishes. Conversely, as release rates increase from 1%, the detrimental impact of the no detention control option on downstream point of interest peak flows would also increase. An implementable watershed-level control philosophy must balance control cost against absolute control effectiveness. Therefore, it is necessary to establish a minimum release rate threshold as part of the Plan. The minimum release rate threshold will be referred to as the breakpoint release rate. The exact location of the breakpoint between where the release rate should be strictly adhered to and where an alternative control should be used cannot be rigorously determined. Rather, it is a fairly subjective breakpoint based on runoff control cost versus the desired level of runoff control (i.e. absolute adherence to the no-increase-in-peak-rate philosophy versus allowing some "insignificant" increase in peak flow in the interest of increased feasibility).

To provide quantitative information for input to the breakpoint determination, an analysis was conducted to determine the incremental cost involved to meet a release rate standard in 10% release rate increments from 100% down to 20%. The analysis was conducted using a "typical" development for the Monocacy Creek Watershed involving medium density residential development on 25 acres and assuming that a detention pond would be used to achieve the desired release rate. The predevelopment condition was assumed to be open space in good condition. Presented in Table 14 is a summary of the results indicating the required detention volume and construction cost, estimated land cost and total cost for each release rate. Also, presented is the cumulative incremental cost of achieving the desired release rate above that required to achieve the 100% release rate control. From Table 14, the detention volume required to achieve the 100% release rate was determined to be 1.4 acre-ft. (approximately 61,000 cubic feet) with a corresponding construction cost of $15,000 excluding land cost. The incremental detention volume required to achieve lower release rates is fairly small and is a flat 0.1 ac-ft/10% release rate for release rates varying from 100% to 60%. Below 60%, the incremental detention volume required is slightly higher at 0.15 ac-ft/10% release rate increment. Overall, there is a 71% increase in the required detention volume between a 100% release rate and a 20% release rate.

Table 14
Release Rate Cost Implications for a "Typical" Development*

Release Rate	Detention Volume Required (acre-ft.)	Construction Cost**	Land Cost***	Total Cost	Cumulative Incremental Cost≠
100%	1.40	$15,000	$25,000	$40,000	$ ---
90%	1.50	$16,000	$26,500	$42,500	$ 2,500
80%	1.60	$17,000	$28,000	$45,000	$ 5,000
70%	1.70	$17,500	$29,500	$47,000	$ 7,000
60%	1.80	$18,500	$31,000	$49,500	$ 9,500
50%	1.95	$19,250	$33,000	$52,250	$12,250
40%	2.10	$20,000	$35,000	$55,000	$15,000
30%	2.25	$21,000	$37,000	$58,000	$18,000
20%	2.40	$22,000	$38,500	$60,500	$20,500

*Source: Dr. Gert Aron (1986 dollars).

**Costs were derived from data for basins constructed in central and western Pennsylvania. Local costs may differ and individual basin costs could differ significantly depending on site characteristics.

***Land cost for 100% release rate basin includes 5-foot water depth with 2-foot freeboard, 2:1 embankment slopes, 10-foot wide berm, 5-foot setback from property line on all four sides, and unit land cost of $50,000 per acre. Land costs for 90% through 20% release rate basins are also based upon 5-foot maximum depth and $50,000 per acre and have been rounded to the nearest $500.

≠Incremental cost to achieve the desired release rate above the cost required to achieve the 100% release rate.

Construction costs associated with the range of release rates are even less variable than the required detention volumes. The incremental cost of providing a 20% release rate relative to a 100% release rate is $7,000, or 47% greater. Across the entire range of release rates, the incremental construction cost per 10% release rate change was very stable (either $1,000 or $750) indicating a uniform economy of scale relative to increasing detention volumes.

The land cost has been developed as a function of the detention basin volume in consideration of typical municipal ordinance design specifications. Land cost for the 100% release rate is based on a five (5) foot water depth, two (2) foot freeboard, ten (10) foot berm width, 2:1 embankment slopes, five (5) foot property line setback and a unit land cost of $50,000 per acre. The 1.4 acre-feet of detention storage for the 100% release rate would therefore require one-half acre of land (rounded to the nearest tenth acre). Additional land costs for storage required above the 100% release rate volume are based upon the same five (5) foot maximum depth and $50,000 per acre unit cost.

The incremental land cost of providing a 20% release rate relative to a 100% release rate is $13,500, an increase of 54%, which is similar to the construction cost impact. Incremental land costs between release rates vary as a function of required volume and have been rounded to the nearest $500.

As is evident from the cumulative incremental cost column, there is no obvious breakpoint release rate from a cost standpoint. The incremental cost by 10% release rate varies from $2,000 to $3,000 or a maximum of approximately 8% per release rate increment (relative to 100% control cost). Release rates as low as 60% can be achieved for less than a 25% increase in total cost relative to the 100% release rate option.

While the information presented in Table 13 is helpful for establishing the detention basin cost impacts of using the release rate philosophy, it does not provide much assistance in determining an appropriate minimum release rate. Therefore, other factors must be considered in establishing the minimum reasonable release rate.

If the cost implications of the control strategy were the overriding concern, a high breakpoint release rate would be used to minimize total runoff control cost. Conversely, if the overriding concern was that peak runoff not be exceeded anywhere in the watershed, then a low breakpoint release rate would be most appropriate. It should be understood that for any subarea with an actual release rate below the breakpoint release rate, any other control strategy implemented will be less effective than the strategy based on meeting the release rate exactly. In other words, any subarea with a sub-breakpoint release rate will have some detrimental impact on the peak flow at some point in the watershed. A counter-balancing impact, however, is that all of the release rates are based upon protection of the most critical drainage point for each subarea which results in over-protection (i.e. reduced peak flows) at many other points.

The conclusion from all of the above is that there is no obvious breakpoint release rate from a cost standpoint and that the only way to determine if a particular breakpoint can achieve the goal of no increase in peak flow throughout the watershed is to establish one, develop the pertinent criteria, and examine the resultant peak flows. To that end, a 50% release rate has been selected as the initial breakpoint based on the following rationale:

- A 50% release rate is the median value and represents a straight compromise between runoff control cost and absolute control effectiveness.
- A 50% breakpoint release rate does not necessarily preclude meeting the no increase in peak goal.

As previously stated, the establishment of a minimum release rate means that each subarea which has a sub-minimum release rate as determined by the runoff model will contribute more runoff at one or more critical drainage points using some other control philosophy (i.e. no detention, a 50% release rate control, etc.). The magnitude of the adverse impact may vary depending upon the control method selected. The minimization of the adverse runoff impact for sub-minimum release rate subareas is an important aspect of the watershed plan and is discussed in the following section.

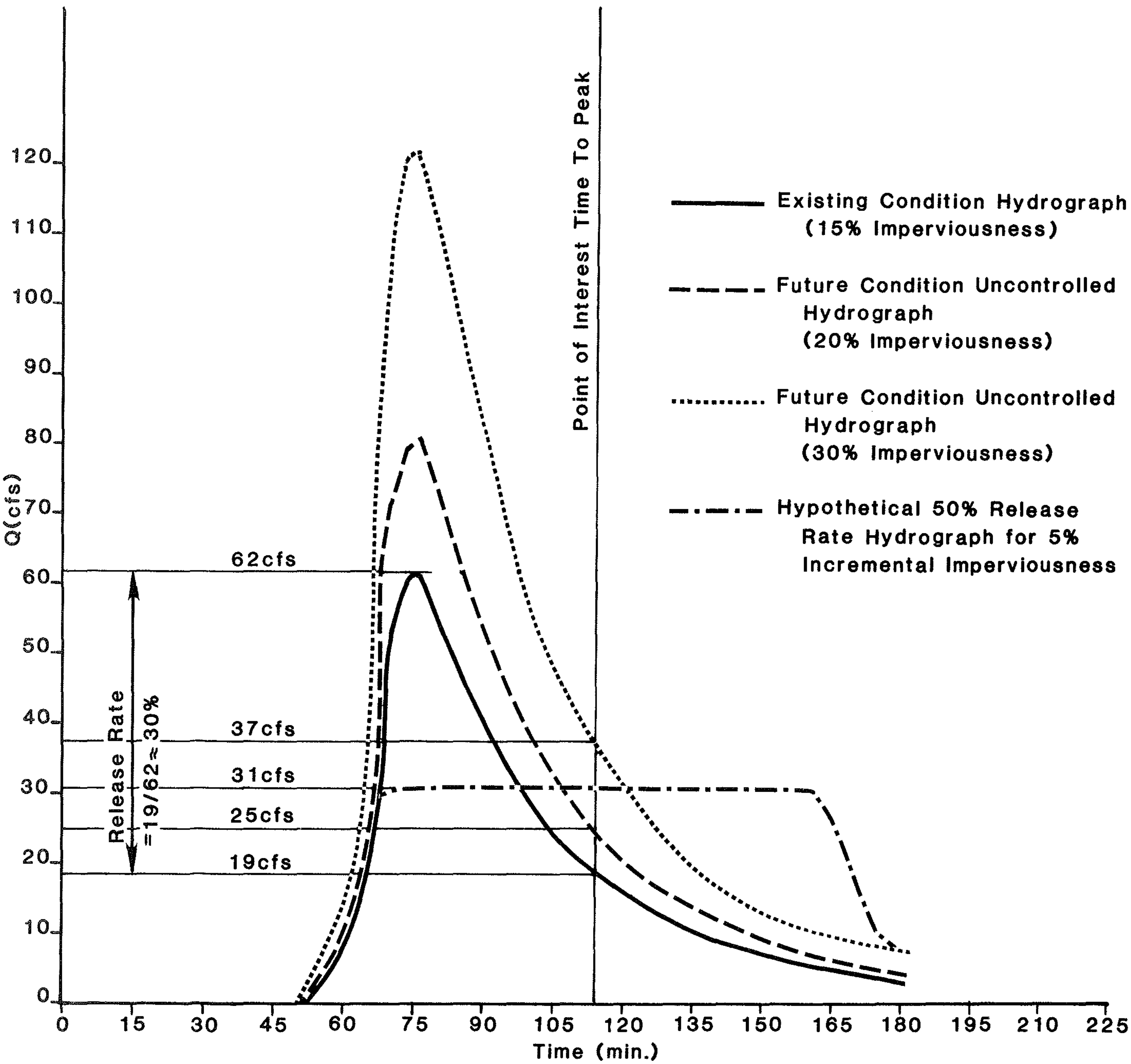
Ultimately, the decision regarding the minimum release rate to be imposed in the watershed depends upon the modeling results. The "gray area" analysis described below is an attempt to find the best runoff control approach using a 50% release rate as an assumed minimum. The final check, however, is to run the model for the future land use condition with the release rate controls in place and test the proposed approach. Should the 50% minimum release rate not achieve the desired runoff control, more stringent criteria would have to be considered.

5. "Gray Area" Analysis

The designation of a minimum reasonable release rate to be used in the development of the watershed-level runoff control strategy creates certain "gray areas" of analysis. The gray areas in the Monocacy Creek Watershed analysis are those areas of the basin for which the hydrologic model generated release rates of less than 50%. For these areas, the most appropriate control strategy would be based on the minimization of any adverse impacts on downstream areas created by not strictly enforcing the actual (sub-50%) release rates. A key aspect of the gray area analysis is identification of an anticipated volume of additional runoff requiring control - i.e. identification of an anticipated future land use condition. An example of this situation is shown in Figure 22. The three hydrographs shown are PSRM-generated hydrographs for identical watersheds with the exception of the amounts of impervious cover assumed. The lowest peaked hydrograph corresponds to 15% imperviousness, the highest peaked hydrograph corresponds to 30% imperviousness, and the in-between hydrograph corresponds to 20% impervious cover as a percentage of the total watershed area. For the purposes of this example, the 15% imperviousness hydrograph will represent the "existing" land use condition and the other two hydrographs will represent two possible future land use conditions. In this example, the time to peak of the point of interest is 115 minutes such that the release rate of the watershed determined by the existing hydrograph is 30%. Note that the uncontrolled post-development runoff for the 20% and 30% imperviousness conditions contribute 25 cfs and 37 cfs, respectively, to the peak at the point of interest compared to 19 cfs in the existing condition.

Since the release rate of the example watershed is 30% based on a point of interest time to peak of 115 minutes, it is clear that using even a 50% release rate philosophy would increase the point of interest peak to a certain extent. The goal of the gray area analysis is to minimize the increase in peak recognizing that the 30% release rate may not be practical. Shown on Figure 22 is the hypothetical outflow hydrograph for the 50% release rate and the future condition percent imperviousness of 20%. Note that the 31 cfs peak outflow extends to nearly 165 minutes before decreasing. The 50% release rate outflow hydrograph for the 30% imperviousness future condition is not shown, but the peak would be extended even past 165 minutes. Minimizing the increase in the point of interest peak essentially means deciding which of two alternatives yields better results. The two alternatives are

FIGURE 22
TYPICAL "GRAY AREA" ANALYSIS



imposing the 50% release rate or not requiring any controls at all. From Figure 22, the contribution of the subject watershed to the point of interest peak for the 20% imperviousness scenario is 25 cfs for the "uncontrolled" post-development condition whereas the 50% release rate condition yields a 31 cfs contribution to the point of interest peak. Therefore, for a 5% increment in percent imperviousness, the preferred alternative would be the "no control" alternative.

Conversely, the 15% incremental imperviousness yields the opposite result (uncontrolled hydrograph contributes 37 cfs and 50% release rate hydrograph contributes 31 cfs).

The conclusion from the above is as follows: If the subject watershed was expected to have a future land use condition consistent with a 5% increment in imperviousness, the no control option would minimize the increase in point of interest peak flow. However, if the future land use condition produces an incremental imperviousness approaching 15%, the preferred option for minimizing the runoff impacts of development at the point of interest would be the 50% release rate option.

The procedure used to determine the most appropriate control strategy for each subarea was as follows:

(a) Run the Penn State Runoff Model for the "existing" condition for the 2-, 10-, 25- and 100- year storms.

(b) For each subarea, determine the most representative release rate across the range of design frequencies.

(c) For subareas with release rates between 50% and 100%, assign the appropriate release rate based upon 10% release rate increments.

(d) For subareas with sub-50% release rates as determined from the existing condition modeling, assign a 50% release rate, 100% release rate, or "no detention" criteria to the subarea based upon the anticipated future land use condition using the "gray area" analysis.

(e) Test the "gray area" analysis by running the model with each subarea from part (d) developed to the future land use condition and with detention basins in place, where applicable, to achieve the assigned release rate.

The most important return period for testing the release rate philosophy as described in part (e) above is the 2-year event. On a percentage basis, the increase in runoff volume between pre- and post-development conditions is greater for the 2-year storm than for any other return period analyzed. This is true because the depth of rainfall is least for the 2-year and the pervious areas (lawns, etc.) do not significantly contribute to peak flows or runoff volume. As the total rainfall depth increases with return period, pervious areas become saturated more quickly and nearly all rainfall becomes runoff -- resembling the response of impervious areas. Therefore, the change in imperviousness with development is more difficult to control from a runoff perspective for the frequent (2-year) storm.

Consistent with the analysis conducted per parts (a) through (d) above, the Monocacy Creek Watershed was divided into various release rate areas and provisional no detention areas. The release rates used varied from 100% down to 50%. For the 2-year return period storm, the optimum release rate configuration resulted in a 20 to 25 percent increase in peak flow in the main channel between the gaging station in Monocacy Park upstream to the Route 512 bridge near Route 22. The maximum increase in peak flow considered acceptable for the analysis was 10% - recognizing that using a 50% (or any other non-zero) minimum release rate would generate some adverse impact on peak flows.

Additional runs of the Monocacy Creek model were made to determine the effectiveness of converting several of the 50% release rate areas to 40%. It was found that lowering the minimum release rate to 40% still generated an increase in peak flow of approximately 15% - or 5% higher than acceptable. Only by lowering the minimum release rate to 30% could the desired control be achieved for the 2-year storm.

A 30% release rate control would represent the most stringent Act 167 criteria developed to-date statewide. The incremental construction cost and land area required to achieve the 30% release rate control (see Table 15), although not exhorbitant, would place an additional burden on a developer. It is important to note, however, that the 30% release rate is required to meet the acceptable peak flow values for the 2-year storm only. Less stringent criteria are necessary for the higher return period events. In fact, only a 100% release rate control is required for the same watershed areas designated for 2-year - 30% when considering storms of return periods of 10 years or above. This is a very important distinction because the cost of a stormwater control facility is predominantly dictated by control of the 100-year storm.

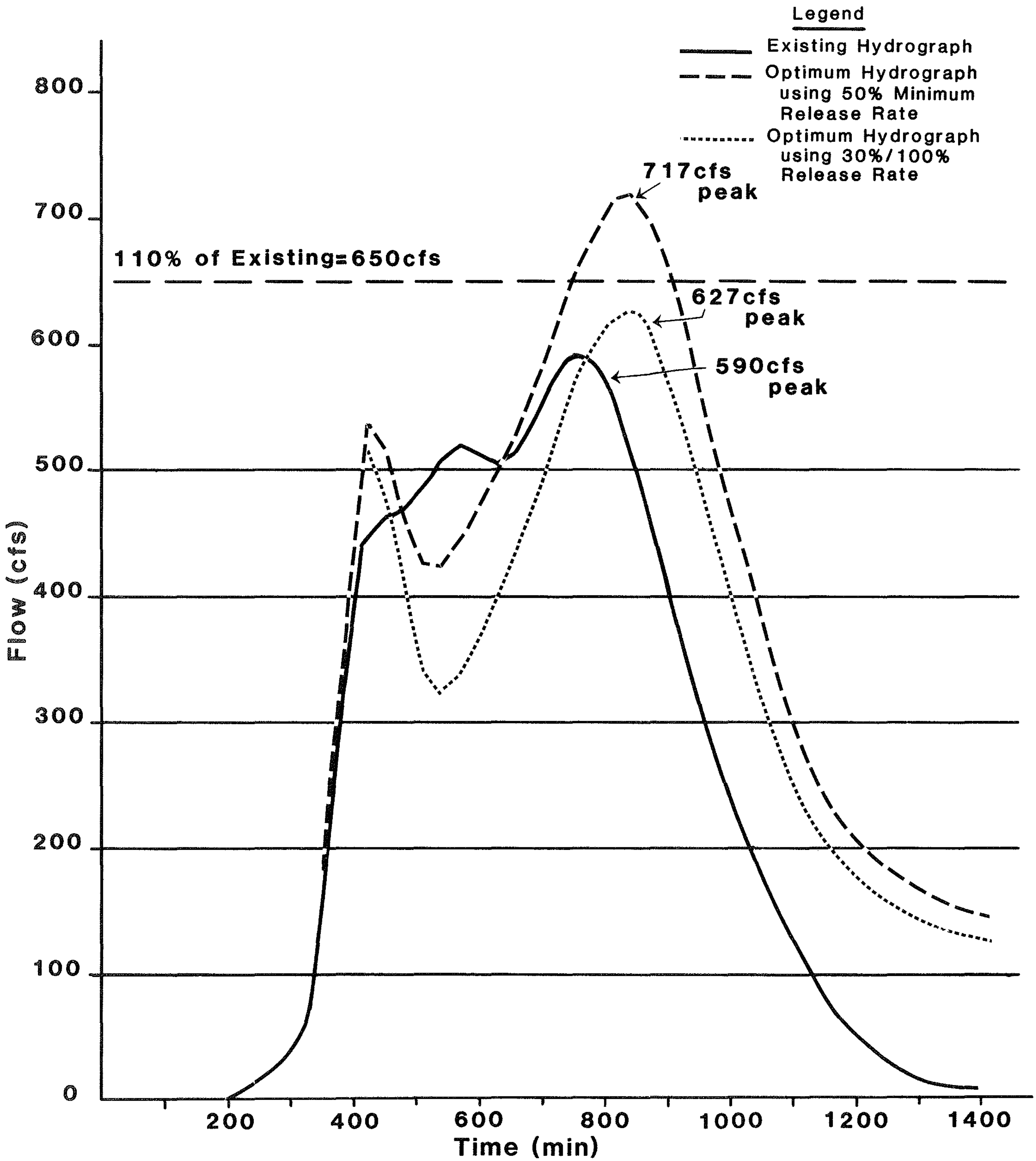
The end product of the above is that a new release rate category has been developed for the Monocacy Creek called the 30%/100% release rate district. Developments within this district would be required to include stormwater controls geared to a 30% release rate for the 2-year storm and a 100% release rate for the 10-, 25- and 100-year storms. This control could be achieved by a fairly minor modification to the design of the outlet structure of a detention pond which would release the 2-year stormflow more slowly, but still meet 100% for the less frequent storms. Therefore, when compared to to 50% release rate district, the 30%/100% release rate district is more effective in controlling the critical storm event (2 year) and is less costly for the developer. The redefinition of certain 50% release rate areas as 30%/100% release rate areas is then doubly beneficial.

Given the advantages of the 30%/100% release rate criteria over the 50% release rate criteria, it would be beneficial to make the 30%/100% area as large as possible. Additional analyses were conducted to determine how far upstream the 30%/100% criteria could be sucessfully applied. The upper limit of the 30%/100% criteria is where "true" 50% release rate areas are encountered. "True" 50% release rate areas are those for which the 50% criteria is required throughout the range of return periods from 2- through 100-years. For "true" 50% release rate areas, attempting to apply the 30%/100% criteria would be beneficial for the 2-year storm, but would create an adverse impact for the higher return periods. "True" release rate areas of 60% through 90% could also not use the 30%/100% criteria for the same reason.

The end product of the "gray area" analysis is the optimum strategy for controlling the potentially adverse impacts on storm runoff from new development while only requiring the level of runoff control which is necessary for a given watershed area. In other words, a developer is only required to control the impacts of his own development activities so as to maintain existing peak flow conditions rather than meet some more stringent arbitrarily chosen control geared towards improving the current runoff situation. It is this fact which maintains the defensibility of the control strategy.

Presented in Figure 23 are several hydrographs developed during the "gray area" analysis to evaluate alternative

FIGURE 23
"Gray Area" Analysis
2-Year Hydrographs at U.S.G.S. Gaging Station in Monocacy Park



control strategies. The three hydrographs presented are the "Existing Condition" 2-year hydrograph at the Monocacy Park gaging station, the optimum 50% Release Rate Minimum hydrograph and the optimum 30%/100% Release Rate hydrograph. Peak flow values increased from the existing condition of 590 cfs to 717 cfs for the 50% Minimum option (21% increase) to only 627 cfs for the 30%/100% Release Rate hydrograph (6% increase). The development of the 30%/100% Release Rate category has therefore resulted in an improvement in control effectiveness of 90 cfs (717-627), or 15% of the existing condition peak flow as compared to the best 50% Minimum Release Rate Strategy.

B. Performance Standards

1. Description of Performance Standard Districts

A major goal of the Act 167 Plan effort was to determine where in the watershed detention is appropriate and, just as importantly, where it is not appropriate for new development. A further goal was to determine to what level of control should detention be provided (i.e. in exchange for an increase in runoff volume with development did existing peak rates need to be reduced). All of the factors described in Section A of this chapter have been incorporated into a control strategy for successfully dealing with the runoff impacts of new development. The control plan is based on dividing the Monocacy Creek Watershed into three basic districts with a finer breakdown in one of the districts. Each of the districts is described below:

(a) Single Release Rate Districts - There are six single release rate districts which differ in the extent to which the post-development runoff must be controlled. The release rates, and districts, are 50%, 60%, 70%, 80%, 90% and 100%. Within a given district, the post-development peak rate of storm runoff must be controlled to the stated percentage of the predevelopment peak rate of runoff for each of the 2-, 10, 25- and 100-year return period storms in order to protect downstream watershed areas.

(b) Provisional No Detention Districts - These watershed areas peak very early with respect to the total watershed peak flow and contribute very minimal flow to the watershed peak flow. For that reason, these watershed areas may discharge post-development peak runoff without detention without adversely affecting the total watershed peak flow. These areas are designated as "provisional" no detention areas because in certain instances the "local" runoff conveyance facilities, which transport runoff from the site to the main channel, may not have adequate capacity to safely transport the peak flows associated with no detention for a proposed development. In those instances, a 100% release rate control would have to be provided or, alternately, the capacity deficiency(ies) would have to a be corrected.

(c) Dual Release Rate Districts - The anticipated post-development runoff from these areas can only be controlled across the range of return periods from 2 through 100 years by implementing a dual system of release rates. This system is designated as 30%/100% release rate criteria. Within this district, the 2 year post-development runoff must be controlled to 30% of the pre-development 2 year runoff peak. Further, the 10-year, 25-year and 100-year post-development runoff must be controlled to 100% of the pre-development peak.

A map of the Monocacy Creek Watershed performance standard districts is included as PLATE I which is located in a map jacket on the inside back cover of the plan.

It is important to emphasize that the release rate criteria (50% to 100% and 30%/100%) represent performance standards for the control of post-development runoff from a development site and not necessarily design criteria for detention facilities. The performance standards may be met with any viable combination of volume controls and rate controls as described in Chapter VI. Volume controls have the benefit of providing for groundwater recharge, but must be implemented carefully to avoid any problems of possible groundwater pollution or aggravation of sinkhole prone areas. The most appropriate control philosophy for a site would be determined only after a thorough site evaluation.

2. Performance Standard Implementation Provisions

The performance standards specified above represent one-half of the storm runoff control strategy for the Monocacy Creek Watershed. The other half of the strategy is composed of the provisions necessary to implement the performance standards including the types of new development to which the standards apply, runoff calculation methodology, criteria for determining downstream channel capacity, a "no harm" procedure for deviating from the performance standards for a particular site and provisions to implement regional detention alternatives. Each of the implementation provisions is addressed separately below.

One additional implementation provision is that the criteria and standards for controlling runoff from new development contained herein are minimum criteria necessary for management of runoff from a watershed perspective. Municipalities may implement more stringent criteria so long as the increased stringency does not conflict with the Plan. A more detailed explanation of this aspect of the Plan is presented in the introduction to the municipal ordinance in Chapter IX.

(a) "New Development" Subject to the Performance Standards

"New development" to be regulated by the runoff control plan would include subdivisions, land developments, construction of new or additional impervious surfaces (driveways, parking lots, etc.), construction of new buildings or additions to existing buildings, diversion or piping of any natural or man-made stream channel, and the installation of any storm sewer systems. The latter two items are included because they may have the impact of modifying significantly the conveyance characteristics which have built into the design of the control plan and, therefore, impact the effectiveness of the control plan. An exemption will be provided in the plan for new developments which are expected to have an insignificant impact on the watershed level runoff characteristics. The exemption will be that any development which will create 10,000 square feet or less of additional impervious cover would not be required to meet the performance standards of the plan. The 10,000 square foot criteria is based on the amount of impervious cover which would generate two (2) cubic feet per second (cfs) or less of additional peak runoff for a five-minute duration storm of an intensity equivalent to a 100-year return period rainfall event. This waiver would not apply to stream channel diversions or piping or storm sewer systems.

(b) Storm Runoff Calculation Methodology

The performance standards will apply to the range of design storm conditions from a 2-year return period to a 100-year return period. This means that the applicable release rates must be met for the 2-year return period storm event, 10-year return period storm event, 25-year return period storm event and 100-year return period storm event. In many instances this will mean that detention facilities would be designed with multiple stage outlet structures to accommodate the range of return periods.

An important implementation provision is the specification of the runoff calculation methods to be used for development sites within the Monocacy Creek Basin. Engineering evaluations of the applicability of various calculation methods were conducted as part of the Plan preparation and supported by previous research. The conclusion from the research is that all development sites in the basin may use either the Rational Method or a soil-cover-complex method for determining pre- and post-development runoff peak rates. The soil-cover-complex method was developed by the Soil Conservation Service (SCS) and its distinguishing characteristic is the use of a parameter called the Runoff Curve Number. SCS has analyzed the runoff relationship between the various land cover and soil type combinations and has formulated a scale of the relative ability of the various combinations to produce runoff from a given rainfall. Although the soil-cover-complex method was developed by SCS, there are many calculation methods available which use the curve number methodology which are not otherwise associated with SCS. The Penn State Runoff Model is one such calculation method.

Regardless of the runoff calculation method used, the design of any detention facility to meet the performance standards specified in the Plan would have to be verified by routing the calculated runoff through the basin. Routing refers to the calculation process of taking the post-development runoff and determining if the detention facility's storage-elevation-outflow characteristics are appropriate for meeting the performance standards. A drawback of the rational method is that it does not provide for routing of flows through a basin. Detention basin volumes may be approximated using the rational (or modified rational) method. However, rational method-derived detention volumes have been found to be consistently lower than those based upon the soil-cover-complex approach. As such, at minimum, any detention facilities within the Monocacy Creek Basin would have to provide a storage volume consistent with a full routing technique. Research conducted for the Plan has found that the approximate routing process contained in Figure 6-1 of SCS Technical Release No. 55 (TR55) results in detention volumes within fifteen percent (15%) of a full routing (i.e. storage indication) technique. Therefore, all detention basins must, at minimum, provide a volume of storage consistent with the approximate routing process contained in Figure 6-1 of TR55. The preferred approach, however, would be the full routing process.

(c) Channel Capacity/Capacity Improvement Criteria

Implementation of the performance standard criteria requires the identification of procedures to deal with two aspects of the "Provisional No Detention" district, namely downstream channel capacity evaluation and possible capacity improvements. The downstream channel capacity analysis is a requirement for the Provisional No Detention areas. Possible channel capacity improvements would be identified as part of a downstream capacity analysis and in certain instances could be implemented in lieu of runoff controls. The procedures involved for each of these implementation aspects is described below.

Proper analysis of channel capacity downstream of a development site for the purpose of discharging greater than predevelopment peak flow rates is essential for ensuring that the goal of not aggravating existing drainage problem areas or creating any new problem areas is achieved. The analysis must include the assumption of development of all areas tributary to the channel being evaluated. The development to be assumed is complete development of the tributary areas based upon reasonable interpretation of existing zoning. Further, all new development within the tributary area must be assumed to implement the appropriate control strategy as specified by the plan. The criteria used to evaluate the adequacy of downstream channel capacity will be as stated below, all three of which must be met to document adequate downstream capacity:

- o Natural or man-made channels must be able to convey the runoff associated with a 2-year return period rainfall event within their banks at velocities consistent with protection of the channels from erosion. Acceptable velocities will be based upon criteria contained in the DER Soil Erosion and Sedimentation Control Manual (Feb. 1985).

- Natural or man-made channels or swales must be able to convey the 25-year return period runoff peak within their banks or otherwise not create any safety or property hazard.

- Culverts, bridges, storm sewers or any other facilities which must pass or convey flows from the tributary area must have sufficient capacity to pass or convey the flows associated with the 25-year return period rainfall event. If, however, the facilities are located within a designated floodplain area per the FIA studies, the adequacy of the facilities shall be based upon the ability to pass or convey the 100-year return period runoff event. Any facilities which qualify as stream enclosures per DER regulations must be capable of conveying the 100-year return period runoff flows.

Any capacity improvements provided in accordance with this Plan would be designed based upon the upstream development assumptions and design criteria as specified for the channel capacity analysis specified above. Capacity improvements would be appropriate where local drainage conditions dictate a more stringent level of control than would watershed-level conditions. The capacity improvements could be provided, therefore, in lieu of runoff control facilities for a development site. This approach has the benefit of minimizing detention facilities provided solely for local reasons. Further, it provides an excellent mechanism for dealing with existing local storm drainage problems caused by existing capacity deficiencies.

(d) "No Harm" Option

The control philosophy as described above incorporating Single Release Rate Districts, Provisional No Detention Districts, Dual Release Rate Districts, downstream capacity analyses and capacity improvements is based on the goal of maintaining (as nearly as possible) existing peak flow values throughout the watershed, or otherwise ensuring that any increase in peak runoff would not adversely impact persons or property. In certain instances, however, the control strategy may be more restrictive than absolutely necessary to achieve the above-stated goal due to special circumstances associated with a given development. For this reason, a "no harm" option is also included as part of the Plan. The purpose for the "no harm" option is to provide a developer with an opportunity to prove that special circumstances exist for his development site which would allow him to deviate from the Plan control strategy, but which would cause "no harm" to persons or property downstream. "Special circumstances" as used above are defined as any hydrologic or hydraulic aspects of the development itself or the downstream conveyance facilities not specifically considered in the development of the Plan runoff control strategy. Two aspects of the Plan runoff control strategy which may particularly provide a developer with a basis for pursuing the "no harm" option are as follows:

(1) The Release Rate strategy is based upon maintaining existing peak rates of flow throughout the watershed after development occurs. In certain instances, the existing drainage network may be capable of safely transporting peak flows in excess of existing flows. A developer may, therefore, be able to prove "no harm" even though peak flows would increase by using a different control strategy than that included in the Plan.

(2) The Release Rate strategy is based on the assumption that the volume of runoff will increase with development of a particular site. In certain instances, however, either due to volume controls proposed by the developer or due to an unusual combination of pre- and post-development conditions, the volume of runoff leaving the site after development may be less than or equal to that prior to development activities. In these instances, it may be possible to discharge peak runoff rates in excess of the Plan criteria without causing harm.

The two key elements of the "no harm" option are that the ability to discharge runoff from a development site at peak rates other than those specified by the Plan would be predicated upon sound engineering proof of "no harm" and that the burden of proof is the responsibility of the developer. To be consistent with the Plan, proof of no harm would have to be shown from the development site through the remainder of the Monocacy Creek Watershed downstream to the confluence with the Lehigh River, since the Plan criteria have been developed consistent with that objective. Conceivably, however, a developer may be able to document the "impact distance" of his proposed actions downstream of which, by definition, no harm would be created. In this way, a developer could limit the downstream extent of the rigorous hydrologic analysis.

Attempts to prove "no harm" based on downstream peak flow versus capacity analysis shall be governed by the following factors:

(1) The peak flow values to be used for downstream areas for various return period storms shall be the values from the calibrated Penn State Runoff Model for the Monocacy Creek Watershed. These flow values would be supplied to the developer by the municipal engineer upon request and are included as Appendix D of the suggested Act 167 ordinance included in Chapter IX.

(2) Any available capacity in the downstream conveyance system as documented by a developer may be used by the developer only in proportion to his development site acreage relative to the total upstream undeveloped acreage from the identified capacity (i.e. if his site is 10% of the upstream undeveloped acreage, he may use up to 10% of the documented downstream available capacity).

(3) Developer-proposed runoff controls which would generate increased peak flow rates at documented storm drainage problem areas would, by definition, be precluded from successful attempts to prove "no harm", except in conjunction with proposed capacity improvements.

The examples of possible bases to pursue "no harm" justifications as presented above are for illustration purposes and are not intended as the only two means available to prove "no harm". It would not be possible to foresee all "special circumstances" of development for which the "no harm" option might be successfully applied. The burden, therefore, would be on the developer to identify the special circumstances and provide the sound engineering "no harm" documentation to the satisfaction of the municipal engineer. "No harm" justifications would be submitted by a developer as part of the Drainage Plan submission included with the Preliminary Plan submission for a subdivision or land development.

(e) Regional or Sub-regional Detention Alternatives

One final aspect of the control philosophy is the provision for regional or sub-regional detention alternatives. The major advantage of a regional facility is the ability to control the runoff from large watershed areas with a single facility rather than one facility for each development site in the tributary area. A single facility would be more aesthetically acceptable than perhaps hundreds of smaller basins and would offer the benefit of much more efficient maintenance.

There are, however, many disadvantages of regional detention facilities. First, regional detention facilities would require large land areas to control large tributary areas. Either the availability of appropriately located land areas or the cost of the land, or both, could preclude this alternative. Second, the financial arrangements for regional facilities may be very cumbersome involving municipal or multi-municipal financing up-front to be reimbursed by developers as the tributary area is developed, as one example. For large tributary areas, the payback time frame would be very uncertain. Third, the design of a regional facility which has tributary areas in multiple control categories specified by this Plan would be complicated. Fourth, the design of a regional facility outlet release would be keyed to protection of the watershed downstream of the regional control. Development upstream of the basin without implementation of on-site runoff controls could create problems between the development site(s) and the basin. This situation would be contradictory to the goals of Act 167.

The above-stated disadvantages of regional detention facilities notwithstanding, it may be feasible to implement regional alternatives within the Monocacy Creek Watershed. The most likely alternatives would involve relatively small tributary areas representing several development sites or possibly one or two subareas. For the purposes of this Plan, any regional alternatives would require the initiative of a developer or group of developers to propose a regional alternative. The funding, design criteria, maintenance provisions and other applicable considerations would be the product of developer-municipal-County discussions. There are no specific recommendations for locations of regional or sub-regional detention facilities incorporated in this Plan. However, as part of the development of the runoff control strategy proposed in the Plan, "sub-regional" detention basins were placed at the outlet of each subarea as delineated for modeling purposes. Acceptable release rates from these basins were determined by running the model several times and varying the release from each basin until the desired post-development hydrograph was achieved for the entire watershed. In the original drawing of the subarea boundaries, an important rationale was to make each subarea small enough such that the hydrograph for each development within the subarea would require the same release rate control as the total subarea. In this way, there would be no difference in design criteria between each development within a subarea and a "sub-regional" facility controlling the entire subarea. Decisions between individual development detention facilities and facilities for entire subareas are therefore dependent upon the type of development(s) proposed and the cost-effectiveness of each control alternative - an evaluation of which is beyond the scope of this report.

Specific locations for regional or sub-regional detention facilities have not been included in this Plan because of the above-described general disadvantages of these facilities and because of the results of the modeling analyses. The optimum location hydrologically for any large-scale regional facilities would be near the confluence of the two major branches in Lower Nazareth Township. However, this area is particularly ill-suited to regional detention facilities because of flat topography and carbonate geology. Facilities at this location would require vast land areas and could be susceptible to sinkhole formation. Since detailed geologic investigations are not part of the Act 167 process, specific locations for regional facilities would be the product of developer-initiated discussions as mentioned above.

CHAPTER IX. MUNICIPAL ORDINANCE TO IMPLEMENT THE MONOCACY CREEK WATERSHED STORM WATER MANAGEMENT PLAN

The implementation of the runoff control strategy for new development will be through municipal adoption of the appropriate ordinance provisions. As part of the preparation of the Monocacy Creek Watershed Storm Water Management Plan, a municipal ordinance has been prepared which would implement the Plan provisions presented in Chapter VIII. The ordinance is a single purpose ordinance which could be adopted essentially as is by the municipalities. Tying provisions would also be required in the municipal Subdivision and Land Development Ordinance and the municipal Building Code to ensure that activities regulated by the ordinance were appropriately referenced. The "Monocacy Creek Watershed Act 167 Storm Water Management Ordinance" will not completely replace the existing storm drainage ordinance provisions currently in effect in the Monocacy Creek municipalities. The reasons for this are as follows:

- o Not all of the municipalities in the Monocacy Creek Basin are completely within the watershed. For those portions of the municipality outside of the Monocacy Creek Watershed, the existing ordinance provisions would still apply.

- o Only permanent storm water control facilities are regulated by the Act 167 Ordinance. Storm water management and erosion and sedimentation control during construction would continue to be regulated under existing laws and ordinances.

- o The Act 167 Ordinance contains only those storm water runoff control criteria and standards which are necessary or desirable from a total watershed perspective. Additional storm water management design criteria (i.e. inlet spacing, inlet type, collection system details, etc.) which should be based on sound engineering practice should be regulated under the current ordinance provisions or as part of the general responsibilities of the municipal engineer.

- o The Act 167 Ordinance contains criteria and standards for runoff control from new development which are the <u>minimum</u> criteria from a watershed perspective. Individual municipalities may adopt more stringent ordinance provisions so long as consistency with the Plan is maintained. Note that more stringent criteria will not always be consistent with the Plan. An example would be a municipality requiring detention for all new development when certain parts of the municipality are within the "Provisional No Detention District."

The Act 167 ordinance is composed of the basic ordinance body and a set of appendices. The body of the document is organized into seven articles as follows:

I - General Provisions
II - Definitions
III - Storm Water Management Requirements
IV - Drainage Plan Requirements
V - Inspections
VI - Fees and Expenses
VII - Maintenance Responsibilities

The Appendices provide maps of the Monocacy Creek Watershed, storm water management districts and storm drainage problem areas as well as technical data to be used in the calculation methodology. The Ordinance is intended to be separable from the Plan document itself. Some of the maps in the Ordinance Appendices would be duplicative of those already included in the Plan and were not included in the Plan version of the Ordinance, but would be included in the separate copies of the Ordinance.

Although the actual storm water control provisions may vary significantly from an existing municipal ordinance, the structure of the ordinance itself is very similar to many existing ordinances. The actual ordinance adopted by a municipality to implement the Monocacy Creek Act 167 Plan may differ in form from the ordinance provided herein so long as it includes, at minimum, all of the provisions of the suggested ordinance. A municipality may tailor the ordinance provisions to best fit into their current ordinance structure. Two notes on the ordinance for municipalities to consider are as follows:

- A "hardship waiver" procedure has been included as Section 407 within Article IV - Drainage Plan Requirements. A municipality may wish to restructure the waiver procedure into a separate Article perhaps as a formal municipal hearing provision. The minimum requirement of the hardship waiver procedure as adopted by a municipality is that it include all four of the "findings" included with the Plan version of the provision.

- The maintenance provisions included in Article VII are structured to eliminate any uncertainty as to the party responsible for continuing maintenance. The elimination of "gray areas" of maintenance responsibilities is the minimum criteria imposed by the ordinance. A municipality may be able to restructure the maintenance provisions to accomplish this minimum goal and place less of a burden on the municipality itself for continuing maintenance.

Presented as the remainder of this chapter is the "Monocacy Creek Watershed Act 167 Storm Water Management Ordinance".

MONOCACY CREEK WATERSHED
ACT 167 STORM WATER MANAGEMENT ORDINANCE

ARTICLE I
GENERAL PROVISIONS

SECTION 101. STATEMENT OF FINDINGS

The governing body of the municipality finds that:

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

B. A comprehensive program of storm water management, including reasonable regulation of development and activities causing accelerated erosion, is fundamental to the public health, safety and 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 the public health, safety and welfare within the Monocacy Creek Watershed by minimizing the damages described in Section 101(A) of this Ordinance by provisions designed to:

A. Control accelerated runoff and erosion and sedimentation problems at their source by regulating activities which cause such problems.

B. Utilize and preserve the desirable existing natural drainage systems.

C. Encourage recharge of groundwaters where appropriate.

D. Maintain the existing flows and quality of streams and water courses in the municipality and the Commonwealth.

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

F. Provide for proper maintenance of all permanent storm water management structures which are constructed in the municipality.

SECTION 103. STATUTORY AUTHORITY

The municipality is empowered to regulate these activities by the authority of the Act of October 4, 1978, P.L. 864 (Act 167), the "Storm Water Management Act" and the (appropriate municipal code).

SECTION 104. APPLICABILITY

This Ordinance shall only apply to those areas of the municipality which are located within the Monocacy Creek drainage basin as delineated on an official map available for inspection at the municipal office. A map of the Monocacy Creek Watershed at a reduced scale is included in Appendix A for general reference.

This Ordinance shall only apply to permanent storm water management facilities constructed as part of any of the activities listed in this section. Storm water management and erosion and sedimentation control during construction involved with any of these activities are specifically not regulated by this Ordinance, but shall continue to be regulated under existing laws and ordinances.

This Ordinance contains only those storm water runoff control criteria and standards which are necessary or desirable from a total watershed perspective. Additional storm water management design criteria (i.e. inlet spacing, inlet type, collection system details, etc.) which represent sound engineering practice may be regulated either by separate storm water ordinance provisions or as part of the general responsibilities of the municipal engineer.

The following activities are defined as Regulated Activities and shall be regulated by this Ordinance, except those which meet the waiver specifications presented thereafter:

A. Land development.

B. Subdivision.

C. Construction of new or additional impervious surfaces (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 storm water systems or appurtenances thereto.

Any proposed Regulated Activity, except those defined in Section 104.E. and 104.F., which would create 10,000 square feet or less of additional impervious cover would be exempt from meeting the provisions of this Ordinance. For development taking place in stages, the entire development plan must be used in determining conformance with this criteria. Additional impervious cover shall include, but not be limited to, any roof, parking or driveway areas and any new streets and sidewalks constructed as part of or for the proposed regulated activity. Any areas which may be designed to initially be semi-pervious (e.g. gravel, crushed stone, porous pavement, etc.) shall be considered impervious areas for the purpose of waiver evaluation. No waiver shall be provided for Regulated Activities as defined in Section 104.E. and 104.F..

SECTION 105. REPEALER

Any ordinance 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 secure required permits or approvals for activities regulated by any other applicable code, rule, act or ordinance.

ARTICLE II
DEFINITIONS

Cistern - An underground reservoir or tank for storing rainwater.

Conservation District - The Northampton County Conservation District (or Lehigh County Conservation District, as applicable).

Culvert - A pipe, conduit or similar structure including appurtenant works which carries surface water.

Design Storm - The magnitude of precipitation from a storm event measured in probability of occurrence (e.g., 50-yr. storm) and duration (e.g. 24-hour), and used in computing storm water management control systems.

Detention Basin - A basin designed to retard storm water 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.

Drainage Easement - A right granted by a land owner to a grantee, allowing the use of private land for storm water management purposes.

Drainage Plan - The documentation of the proposed storm water management controls, if any, to be used for a given development site, the contents of which are established in Section 403.

Erosion - The removal of soil particles by the action of water, wind, ice, or other geological agents.

Freeboard - The incremental depth in a storm water management structure, provided as a safety factor of design, above that required to convey the design runoff event.

Groundwater Recharge - Replenishment of existing natural underground water supplies.

Impervious Surface - A surface which prevents the percolation of water into the ground.

Infiltration Structure - A structure designed to direct runoff into the ground, e.g. french drain, seepage pit or seepage trench.

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) a subdivision of land.

Mainstem (main channel) - Any stream segment or other runoff conveyance facility used as a reach in the Monocacy Creek hydrologic model.

Manning Equation (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.

Peak Discharge - The maximum rate of flow of storm runoff at a given point and time resulting from a specified storm event.

Penn State Runoff Model (calibrated) - The computer-based hydrologic modeling technique adapted to the Monocacy Creek Watershed for the Act 167 Plan. The model has been "calibrated" to reflect actual recorded flow values by adjusting key model input parameters.

Rational Method - A method of peak runoff calculation using a standardized runoff coefficient (rational 'c'), acreage of tract and rainfall intensity determined by return period and by the time necessary for the entire tract to contribute runoff. The rational formula is stated as follows: Q = ciA, where "Q" is the calculated peak flow rate in cubic feet per second, "c" is the dimensionless runoff coefficient (see Appendix C), "i" is the rainfall intensity in inches per hour, and "A" is the area of the tract in acres.

Reach - Any of the 101 natural or man-made runoff conveyance channels used for modeling purposes to connect the subareas and transport flows downstream.

Regulated Activities - Actions or proposed actions which impact upon proper management of storm water runoff and which are governed by this Ordinance as specified in Section 104.

Release Rate - The percentage of the predevelopment peak rate of runoff for a development site to which the post-development peak rate of runoff must be controlled to protect downstream areas.

Return Period - The average interval in years over which an event of a given magnitude can be expected to recur. For example, the twenty-five (25) year return period rainfall or runoff event would be expected to recur on the average once every twenty-five years.

Runoff - That part of precipitation which flows over the land.

SCS - Soil Conservation Service, U.S. Department of Agriculture.

Seepage Pit/Seepage Trench - An area of excavated earth filled with loose stone or similar material and into which surface water is directed for infiltration into the ground.

Soil-Cover-Complex Method - A method of runoff computation developed by SCS which is based upon relating soil type and land use/cover to a runoff parameter called a Curve Number.

Storage Indication Method - A reservoir routing procedure based on solution of the continuity equation (inflow minus outflow equals the change in storage for a given time interval) and based on outflow being a unique function of storage volume.

Storm Sewer - A system of pipes or other conduits which carries intercepted surface runoff, street water and other wash waters, or drainage, but excludes domestic sewage and industrial wastes.

Storm Water Management Plan - The plan for managing storm water runoff adopted by Northampton County for the Monocacy Creek Watershed as required by the Act of October 4, 1978, P.L. 864, (Act 167), and known as the "Storm Water Management Act".

Stream - A watercourse.

Subarea - The smallest unit of watershed breakdown for hydrologic modeling purposes for which the runoff control criteria have been established in the Storm Water Management Plan.

Subdivision - The division or redivision 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.

Swale - A low lying stretch of land which gathers or carries surface water runoff.

Watercourse - Any channel of conveyance of surface water having defined bed and banks, whether natural or artificial, with perennial or intermittant flow.

ARTICLE III
STORM WATER MANAGEMENT REQUIREMENTS

SECTION 301. GENERAL REQUIREMENTS

A. Storm drainage systems shall be provided in order to permit unimpeded flow of natural watercourses except as modified by storm water detention facilities or open channels consistent with this Ordinance.

B. The existing points of concentrated drainage discharge onto adjacent property shall not be altered without written approval of the affected property owner(s).

C. Areas of existing diffused drainage discharge onto adjacent property shall be managed such that, at minimum, the peak diffused flow does not increase in the general direction of discharge, except as otherwise provided in this Ordinance. If diffused flow is proposed to be concentrated and discharged onto adjacent property, the developer must document that there are adequate downstream conveyance facilities to safely transport the concentrated discharge or otherwise prove that no harm will result from the concentrated discharge. Areas of existing diffused drainage discharge shall be subject to any applicable release rate criteria in the general direction of existing discharge whether they are proposed to be concentrated or maintained as diffused drainage areas.

D. Where a subdivision is traversed by watercourses other than permanent streams, there shall be provided a drainage easement conforming substantially with the line of such watercourse. The width of the easement shall be adequate to provide for unimpeded flow of storm runoff based on calculations made in conformance with Section 304 for the 100-year return period runoff and to provide a freeboard allowance of one-half (0.5) foot above the design water surface level. The terms of the easement shall prohibit excavation, the placing of fill or structures, and any alterations which may adversely affect the flow of storm water within any portion of the easement. Also, periodic maintenance of the easement to ensure proper runoff conveyance shall be required.

E. Any drainage facilities required by this Ordinance that are located on State highway rights-of-way shall be subject to approval by the Pennsylvania Department of Transportation.

F. When it can be shown that, due to topographic conditions, natural drainage swales on the site cannot adequately provide for drainage, open channels may be constructed conforming substantially to the line and grade of such natural drainage swales. Capacities of open channels shall be calculated using the Manning equation.

G. Storm drainage facilities and appurtenances shall be so designed and provided as to minimize erosion in watercourse channels and at all points of discharge.

H. Consideration should be given to the design and use of volume controls for storm water management, where geology permits.

SECTION 302. STORM WATER MANAGEMENT DISTRICTS

A. Mapping of Storm Water Management Districts - In order to implement the provisions of the Monocacy Creek Storm Water Management Plan, the municipality is hereby divided into Storm Water Management Districts consistent with the Monocacy Creek Release Rate Map presented in the Plan. The boundaries of the Storm Water Management Districts are shown on an official map which is available for inspection at the municipal office. A copy of the official map at a reduced scale is included in Appendix A for general reference.

B. Description of Storm Water Management Districts - Three types of Storm Water Management Districts may be applicable to the municipality, namely Single Release Rate Districts, Provisional No Detention Districts and Dual Release Rate Districts as described below.

   1. Single Release Rate Districts - There are six single release rate districts which differ in the extent to which post-development runoff must be controlled. The release rates, and districts, are 50%, 60%, 70%, 80%, 90% and 100%. Within a given district, the post-development peak rate of storm runoff must be controlled to the stated percentage of the predevelopment peak rate of storm runoff in order to protect downstream watershed areas.

   2. Provisional No Detention Districts - These watershed areas may discharge post-development peak runoff without detention facilities without adversely affecting the total watershed peak flow. In certain instances, however, the "local" runoff conveyance facilities, which transport runoff from the site to the main channel, may not have adequate capacity to safely transport increased peak flows associated with not providing detention for a

proposed development. In those instances, the developer shall either use a 100% release rate control or provide increased capacity of downstream drainage elements to convey increased peak flows consistent with Section 303.H. In determining if adequate capacity exists in the local watershed drainage network, the developer must assume that the entire local watershed is developed per current zoning and that all new development would use the runoff controls specified by this Ordinance. Similarly, any capacity improvement must be designed to convey runoff from development of all areas tributary to the improvement consistent with the capacity criteria specified in Section 303.C.

3. Dual Release Rate Districts - The anticipated post-development runoff from these areas can only be controlled across the range of return periods from 2 through 100 years by implementing a dual system of release rates. This system is designated as 30%/100% release rate criteria. Within this district, the 2-year post-development runoff must be controlled to 30% of the pre-development 2-year runoff peak. Further, the 10-year, 25-year and 100-year post-development runoff must be controlled to 100% of the predevelopment peak.

SECTION 303. STORM WATER MANAGEMENT DISTRICT IMPLEMENTATION PROVISIONS

A. Any storm water management controls required by this Ordinance and subject to single release rate criteria (50% through 100%) shall meet the applicable release rate criteria for each of the 2-, 10-, 25- and 100-year return period runoff events consistent with the calculation methodology specified in Section 304. Storm water management controls intended to meet the Dual Release Rate (30%/100%) criteria shall also be designed consistent with Section 304.

B. The exact location of the Storm Water Management District boundaries as they apply to a given development site shall be determined by mapping the boundaries using the two-foot topographic contours provided as part of the Drainage Plan. The District boundaries as originally drawn coincide with topographic divides or, in certain instances, are drawn from the intersection of the watercourse and a physical feature (such as the confluence with another watercourse or a potential flow obstruction e.g. road, culvert, bridge, etc.) to the topographic divide consistent with topography.

C. Any downstream capacity analysis conducted in accordance with this Ordinance 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 increased 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 DER Soil Erosion and Sedimentation Control Manual (February, 1985) and presented in Appendix C of this Ordinance.

2. Natural or man-made channels or swales must be able to convey the increased 25-year return period runoff peak within their banks or otherwise not create 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 have sufficient capacity to pass or convey the increased flows associated with the 25-year return period runoff event, except for facilities located within a designated floodplain area which must be capable of passing or conveying the 100-year return period runoff. Any facilities which constitute stream enclosures per DER's Chapter 105 regulations shall be designed to convey the 100-year return period runoff.

D. For a proposed development site located within only one release rate category area, the total runoff from the site shall meet the applicable release rate criteria. For development sites with multiple points of concentrated runoff discharge, individual drainage points may be designed for up to a 100% release rate so long as the total runoff from the site is controlled to the applicable release rate.

E. For a proposed development site located within two or more release rate category areas, the maximum peak rate of runoff that may be discharged at any point is limited to the predevelopment peak rate of runoff at that point multiplied by the applicable release rate. The control rates shall apply regardless of any grading modifications which may change the drainage area which discharges at a given point.

F. For proposed development sites located partially within a release rate category area and partially within a provisional no detention area, in no event shall a significant portion of the site area subject to the release rate control be drained to the discharge point(s) located in the no detention area.

G. "No Harm" Option - For any proposed development site not located in a provisional no detention district, the developer has the option of using a less restrictive runoff control (including no detention) if the developer can prove that "no harm" would be caused by discharging at a higher runoff rate than that specified by the Plan. Proof of "no harm" would have to be shown from the development site through the remainder of the downstream drainage network to the confluence of the Monocacy Creek with the Lehigh River. Proof of "no harm" must be shown using the capacity criteria specified in Section 303.C. if downstream capacity analysis is a part of the "no harm" justification.

Attempts to prove "no harm" based upon downstream peak flow versus capacity analysis shall be governed by the following provisions:

1. The peak flow values to be used for downstream areas for the design return period storms (2-, 10-, 25- and 100-year) shall be the values from the calibrated Penn State Runoff Model for the Monocacy Creek Watershed. These flow values would be supplied to the developer by the municipal engineer upon request.

2. Any available capacity in the downstream conveyance system as documented by a developer may be used by the developer only in proportion to his development site acreage relative to the total upstream undeveloped acreage from the identified capacity (i.e. if his site is 10% of the upstream undeveloped acreage, he may use up to 10% of the documented downstream available capacity).

3. Developer-proposed runoff controls which would generate increased peak flow rates at documented storm drainage problem areas would, by definition, be precluded from successful attempts to prove "no harm," except in conjunction with proposed capacity improvements for the problem areas consistent with Section 303.I.

Any "no harm" justifications shall be submitted by the developer as part of the Drainage Plan submission per Article IV.

H. Regional or Sub-Regional Detention Alternatives - For certain areas within the watershed, it may be more cost-effective to provide one control facility for an entire subarea, group of subareas, or portion of a subarea incorporating more than one development site than to provide an individual control facility for each development site. The initiative and funding for any regional or sub-regional runoff control alternatives are the responsibility of prospective developers. The design of any regional control basins must incorporate reasonable development of the entire upstream watershed. The peak outflow of a regional basin would be determined on a case-by-case basis using the hydrologic model of the watershed consistent with protection of the downstream watershed areas. "Hydrologic model" refers to the calibrated Monocacy Creek version of the Penn State Runoff Model as developed for the Storm Water Management Plan.

I. Capacity Improvements - In certain instances, primarily within the provisional no detention areas, local drainage conditions may dictate more stringent levels of runoff control than those based upon protection of the entire watershed. In these instances, if the developer could prove that it would be feasible to provide capacity improvements to relieve the capacity deficiency in the local drainage network, then the capacity improvements could be provided by the developer in lieu of runoff controls on the development site. Any capacity improvements would be designed based upon development of all areas tributary to the proposed improvement and the capacity criteria specified in Section 303.C. In addition, all new development upstream of a proposed capacity improvement shall be assumed to implement the applicable runoff controls consistent with this Ordinance except that all new development within the entire subarea(s) within which the proposed development site is located shall be assumed to implement the developer's proposed discharge control, if any.

Capacity improvements may also be provided as necessary to implement any regional or subregional detention alternatives or to implement a modified "no harm" option which proposes specific capacity improvements to document the validity of a less stringent discharge control which would not create any harm downstream.

J. Waiver of Runoff Control Based On Minimum Additional Impervious Cover - Any proposed Regulated Activity, except those defined in Sections 104.E. and 104.F., which would create 10,000 square feet or less of additional impervious cover would be exempt from meeting the runoff control provisions of this Ordinance. For developments which are to take place in stages, the entire development plan must be used in determining conformance to this criteria. Additional impervious cover shall include, but not be limited to, any roof, parking or driveway areas and any new streets and sidewalks constructed as part of or for the proposed development. Any areas which may be designed to initially be semi-pervious (e.g. gravel, crushed stone, porous pavement, etc.) shall be considered impervious areas for the purposes of waiver evaluation.

No waiver shall be provided for any Regulated Activities as defined in Sections 104.E. and 104.F.

SECTION 304. CALCULATION METHODOLOGY

A. Storm water runoff from all development sites shall be calculated using either the rational method or a soil-cover-complex methodology.

B. The design of any detention basin intended to meet the requirements of this Ordinance shall be verified by routing the design storm hydrograph through the proposed basin. For basins designed using the modified rational method technique, the detention volume shall, at minimum, equal the volume derived from the approximate routing process as contained in SCS Technical Release Number 55 (TR55, 1986), Chapter 6 (Figure 6-1).

C. All storm water detention facilities shall provide a minimum 1.0 foot freeboard above the maximum pool elevation associated with the 2- through 25-year runoff events. An emergency spillway shall be designed to pass the 100-year runoff event with a minimum 0.5 foot freeboard.

D. All calculations using the soil-cover-complex method shall use the Soil Conservation Service Type II 24-hour rainfall distribution. The 24-hour rainfall depths for the various return periods to be used consistent with this Ordinance are taken from the PennDOT Intensity - Duration - Frequency Field Manual (May 1986) for Region 4:

Return Period	24-Hour Rainfall Depth
2 year	2.88 inches
10 year	4.56 inches
25 year	5.52 inches
100 year	7.68 inches

A graphical and tabular presentation of the Type II-24 hour distribution is included in Appendix C.

E. All calculations using the Rational Method shall use rainfall intensities consistent with appropriate times of concentration and return periods and the Intensity-Duration - Frequency Curves as presented in Appendix C.

F. Runoff Curve Numbers (CN's) to be used in the soil-cover-complex method shall be based upon the matrix presented in Appendix C.

G. Runoff coefficients for use in the Rational Method shall be based upon the table presented in Appendix C.

H. The Manning equation shall be used to calculate the capacity of watercourses. Manning 'n' values used in the calculations shall be consistent with the table presented in Appendix C. Pipe capacities shall be determined by methods acceptable to the municipal engineer.

I. Any detention basin intended to meet the requirements of this Ordinance which requires a Dam Safety Permit from DER shall be designed consistent with the provisions of the Dam Safety and Encroachments Act and the DER Chapter 105 Rules and Regulations.

ARTICLE IV
DRAINAGE PLAN REQUIREMENTS

SECTION 401. GENERAL REQUIREMENTS

For any of the Regulated Activities of this Ordinance, prior to the final approval of subdivision and/or land development plans, or the issuance of any permit, or the commencement of any land disturbance activity, the owner, subdivider, developer or his agent shall submit a Drainage Plan for approval.

SECTION 402. EXEMPTIONS

Any Regulated Activity which would create 10,000 square feet or less of additional impervious cover is exempt from the Drainage Plan preparation provisions of this Ordinance. This criteria shall apply to the total proposed development even if development is to take place in stages (i.e. the impervious cover associated with the total development shall be used to compare to the waiver minimum, not merely the individual stage impervious cover). Additional impervious cover shall include, but not be limited to, any roof, parking or driveway areas and any new streets and sidewalks constructed as part of or for the proposed Regulated Activity. Any areas designed to initially be gravel, crushed stone, porous pavement, etc. shall be assumed to be impervious for the purposes of comparison to the waiver criteria.

SECTION 403. DRAINAGE PLAN CONTENTS

The following items shall be included in the Drainage Plan:

A. General

1. General description of project.

2. General description of proposed permanent storm water controls.

B. Map(s) of the project area showing:

1. The location of the project relative to highways, municipalities or other identifiable landmarks.

2. Existing contours at intervals of two (2) feet. In areas of steep slopes (greater than 15%), five-foot contour intervals may be used.

3. Streams, lakes, ponds or other bodies of water within the project area.

4. Other physical features including existing drainage swales and areas of natural vegetation to be preserved.

5. Locations of proposed underground utilities, sewers and water lines.

6. An overlay showing soil types and boundaries.

7. Proposed changes to land surface and vegetative cover.

8. Proposed structures, roads, paved areas and buildings.

9. Final contours at intervals of two (2) feet. In areas of steep slopes (greater than 15%), five-foot contour intervals may be used.

10. Storm Water Management District boundaries applicable to the site.

C. Storm water management controls

1. All storm water management controls must be shown on a map and described, including:

a. Groundwater recharge methods such as seepage pits, beds or trenches. When these structures are used, the locations of septic tank infiltration areas and wells must be shown.

b. Other control devices or methods such as roof-top storage, semi-pervious paving materials, grass swales, parking lot ponding, vegetated strips, detention or retention ponds, storm sewers, etc..

2. All calculations, assumptions and criteria used in the design of the control device or method must be shown.

D. Maintenance Program - A maintenance program for all storm water management control facilities must be included. This program must include the proposed ownership of the control facilities, the maintenance requirements for the facilities, and the financial responsibilities for the required maintenance.

SECTION 404. PLAN SUBMISSION

A. For Regulated Activities specified in Sections 104.A. and 104.B.:

1. The Drainage Plan shall be submitted by the developer to the municipal secretary (or other appropriate person) as part of the Preliminary Plan submission for the subdivision or land development.

2. Three (3) copies of the Drainage Plan shall be submitted.

3. Distribution of the Drainage Plan will be as follows:

a) One (1) copy to the municipal governing body.

b) One (1) copy to the municipal engineer.

c) One (1) copy to the Joint Planning Commission.

B. For Regulated Activities specified in Sections 104.C. and 104.D., the Drainage Plan shall be submitted by the developer to the municipal building permit officer as part of the building permit application.

C. For Regulated Activities specified in Sections 104.E. and 104.F.:

1. The Drainage Plan shall be submitted by the developer to the Joint Planning Commission for coordination with the DER permit application process under Chapter 105 (Dam Safety and Waterway Management) or Chapter 106 (Flood Plain Management) of DER's Rules and Regulations.

2. One (1) copy of the Drainage Plan shall be submitted.

SECTION 405. DRAINAGE PLAN REVIEW

A. The municipal engineer shall review the Drainage Plan for consistency with the adopted Monocacy Creek Storm Water Management Plan as embodied by this ordinance and against any additional storm drainage provisions contained in the municipal subdivision and land development or zoning ordinance, as applicable.

B. The Joint Planning Commission shall provide an advisory review of the Drainage Plan for consistency with the Monocacy Creek Storm Water Management Plan.

C. For Regulated Activities specified in Sections 104.A. and 104.B., the JPC shall provide written comments to the municipality, within a time frame consistent with established procedures under Act 247, as to whether the Drainage Plan has been been found to be consistent with the Storm Water Management Plan.

D. For Regulated Activities specified in Sections 104.E. and 104.F., the JPC shall notify DER whether the Drainage Plan is consistent with the Storm Water Management Plan and forward a copy of the review letter to the municipality and developer.

E. The municipality shall not approve any subdivision or land development (Regulated Activities 104.A. and 104.B.) or building permit application (Regulated Activities 104.C. and 104.D.) if the Drainage Plan has been found to be inconsistent with the Storm Water Management Plan as determined by the municipal engineer.

SECTION 406. MODIFICATION OF PLANS

A modification to a submitted Drainage Plan for a proposed development site which involves a change in control methods or techniques, or which involves the relocation or redesign of control measures, or which 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 subject to review per Section 405 of this Ordinance.

SECTION 407. HARDSHIP WAIVER PROCEDURE

The municipality (governing body) may hear requests for waivers where it is alleged that the provisions of this (Act 167) Ordinance inflict unnecessary hardship upon the applicant. The waiver request shall be in writing on an application form promulgated by the municipality and accompanied by the requisite fee based upon a fee schedule adopted by the municipality. A copy of the completed application form shall be provided to each of the following: municipality, municipal engineer, municipal solicitor and Joint Planning Commission. The application shall fully document the nature of the alleged hardship.

The municipality may grant a waiver provided that all of the following findings are made in a given case:

1. That there are unique physical circumstances or conditions, including irregularity of lot size or shape, or exceptional topographical or other physical conditions peculiar to the particular property, and that the unnecessary hardship is due to such conditions, and not the circumstances or conditions generally created by the provisions of this Ordinance in the Storm Water Management District in which the property is located;

2. That because of such physical circumstances or conditions, there is no possibility that the property can be developed in strict conformity with the provisions of this Ordinance, including the "no harm" provision, and that the authorization of a waiver is therefore necessary to enable the reasonable use of the property;

3. That such unnecessary hardship has not been created by the applicant; and

4. That the waiver, if authorized, will represent the minimum waiver that will afford relief and will represent the least modification possible of the regulation in issue.

In granting any waiver, the municipality (governing body) may attach such reasonable conditions and safeguards as it may deem necessary to implement the purposes of Act 167 and this Ordinance.

ARTICLE V
INSPECTIONS

SECTION 501. SCHEDULE OF INSPECTIONS

A. The municipal engineer or his designee shall inspect all phases of the installation of the permanent storm water control facilities and the completed installation.

B. If at any stage of the work the municipal engineer determines that the permanent storm water control facilities are not being installed in accordance with the approved development plan, the municipality shall revoke any existing permits until a revised development plan is submitted and approved as required by Section 406.

ARTICLE VI
FEES AND EXPENSES

SECTION 601. GENERAL

A fee shall be established by the municipality to defer municipal costs for Drainage Plan review and processing.

SECTION 602. EXPENSES COVERED BY FEES

The fees required by this Ordinance shall at a minimum cover:

A. The review of the Drainage Plan by the municipal engineer.

B. The site inspection.

C. The inspection of required controls and improvements during construction.

D. The final inspection upon completion of the controls and improvements required in the plan.

E. Any additional work required to enforce any permit provisions, regulated by this Ordinance, correct violations, and assure the completion of stipulated remedial actions.

ARTICLE VII
MAINTENANCE RESPONSIBILITIES

SECTION 701. MAINTENANCE RESPONSIBILITIES

The maintenance responsibilities for permanent storm water runoff control facilities shall be determined based upon the type of ownership of the property which is controlled by the facilities.

A. Single Entity Ownership - In all cases where the permanent storm water runoff control facilities are designed to manage runoff from property in a single entity ownership as defined below, the maintenance responsibility for the storm water control facilities shall be with the single entity owner. The single entity owner shall enter into an agreement with the municipality which specifies that the owner will properly maintain the facilities consistent with accepted practice as determined by the municipal engineer. The agreement shall provide for regular inspections by the municipality and contain such provisions as necessary to ensure timely correction of any maintenance deficiencies by the single entity owner. A single entity shall be defined as an individual, association, public or private corporation, partnership firm, trust, estate or any other legal entity empowered to own real estate.

B. Multiple Ownership - In cases where the property controlled by the permanent storm water control facilities shall be in multiple ownership (i.e. many individual owners of various portions of the property), the developer shall dedicate the permanent storm water control facilities to the municipality for maintenance. The developer shall pay a fee to the municipality corresponding to the present worth of maintenance of the facilities for a ten-year period. The estimated annual maintenance cost for the facilities shall be based on a fee schedule provided by the municipal engineer and adopted by the municipality. The fee schedule must be reasonable.

In certain multiple ownership situations, the municipality may benefit by transferring the maintenance responsibility to an individual or group of individuals residing within the controlled area. These individuals may have the permanent storm water control facilities adjacent to their lots or otherwise have an interest in the proper maintenance of the facilities. In these instances, the municipality and the individual(s) may enter into a formal agreement for the maintenance of the facilities. The municipality shall maintain ownership of the facilities and be responsible for periodic inspections.

SECTION 702. RIGHT-OF-ENTRY

Upon presentation of the proper credentials, duly authorized representatives of the municipality may enter at reasonable times upon any property within the municipality to investigate or ascertain whether proper maintenance is being provided for any storm water management facilities for which the municipality is not directly responsible for maintenance as provided in Section 701.

APPENDIX A
(Not Included in Plan Copy of Ordinance)

A-1 Map of Monocacy Creek Watershed

A-2 Municipal Map of Storm Water Management Districts

APPENDIX B

(Not Included in Plan Copy of Text)

B-1 Map of Storm Drainage Problem Areas

B-2.. Description of Storm Drainage Problem Areas

APPENDIX C

C-1 SCS Type II 24-Hour Rainfall Distribution (Graphic & Tabular)

C-2 Intensity-Duration-Frequency Curves

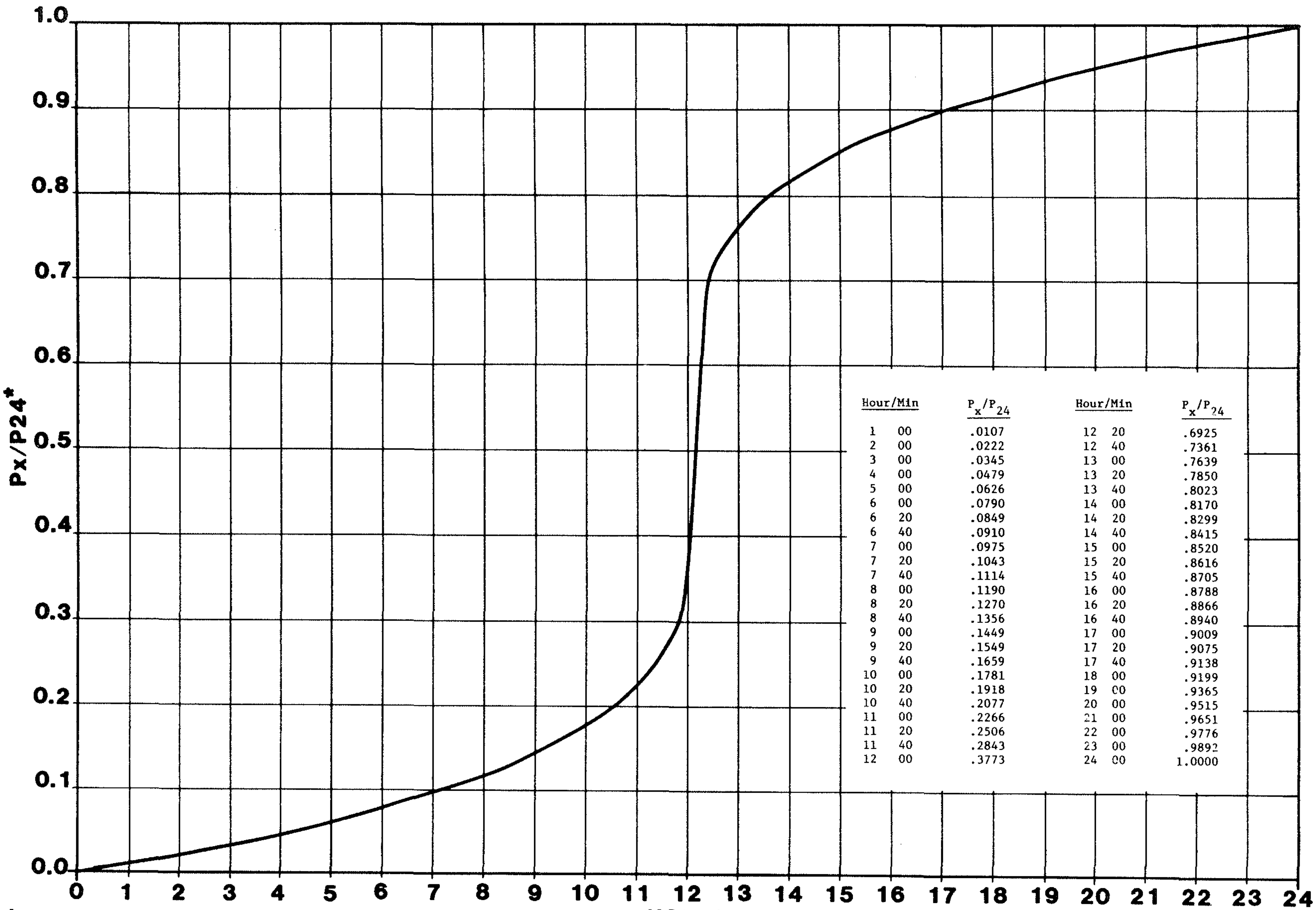
C-3 Runoff Curve Numbers and Percent Imperviousness Values

C-4, 5 Runoff Coefficients for the Rational Method

C-6 Manning 'n' Values

C-7 Permissible Velocities for Channels

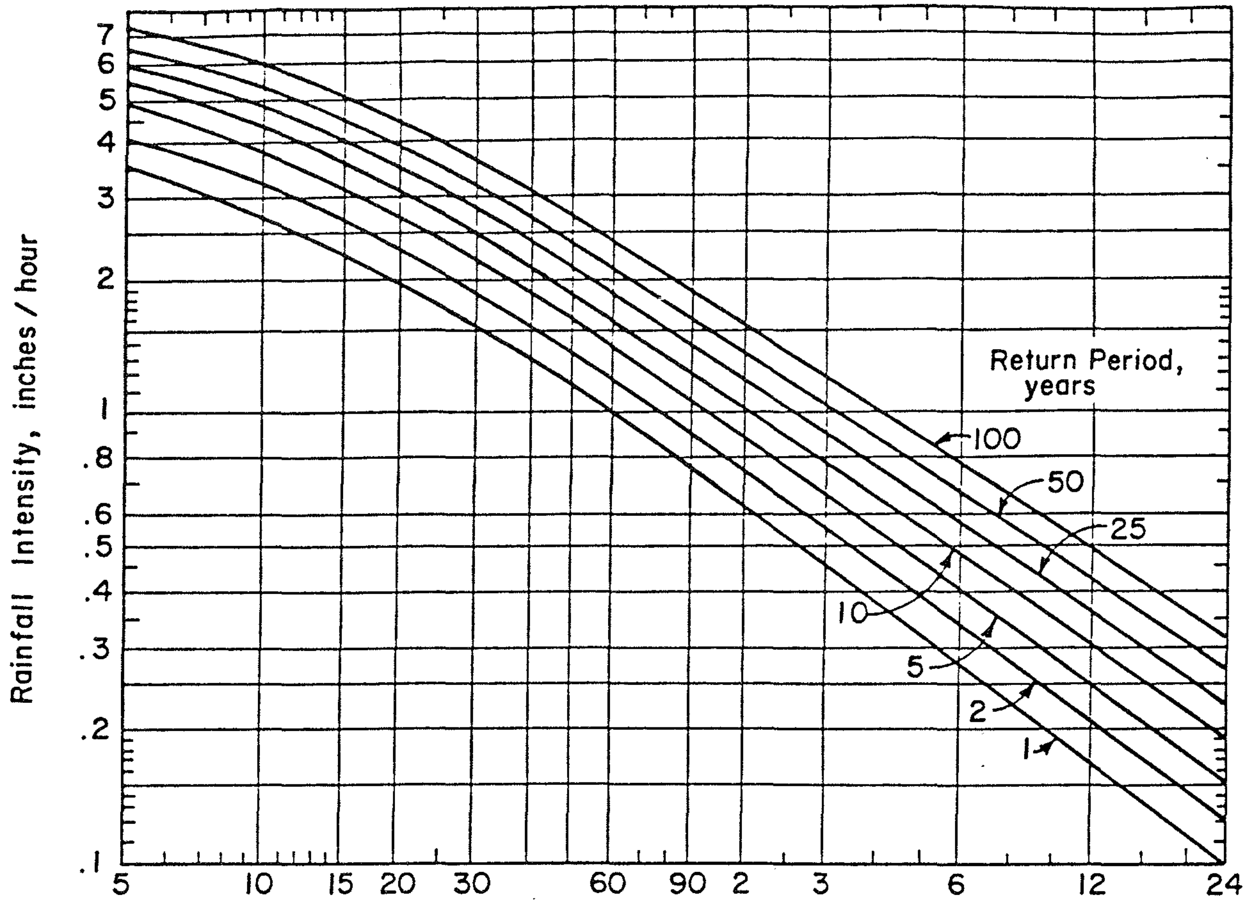
SCS TYPE II RAINFALL DISTRIBUTION

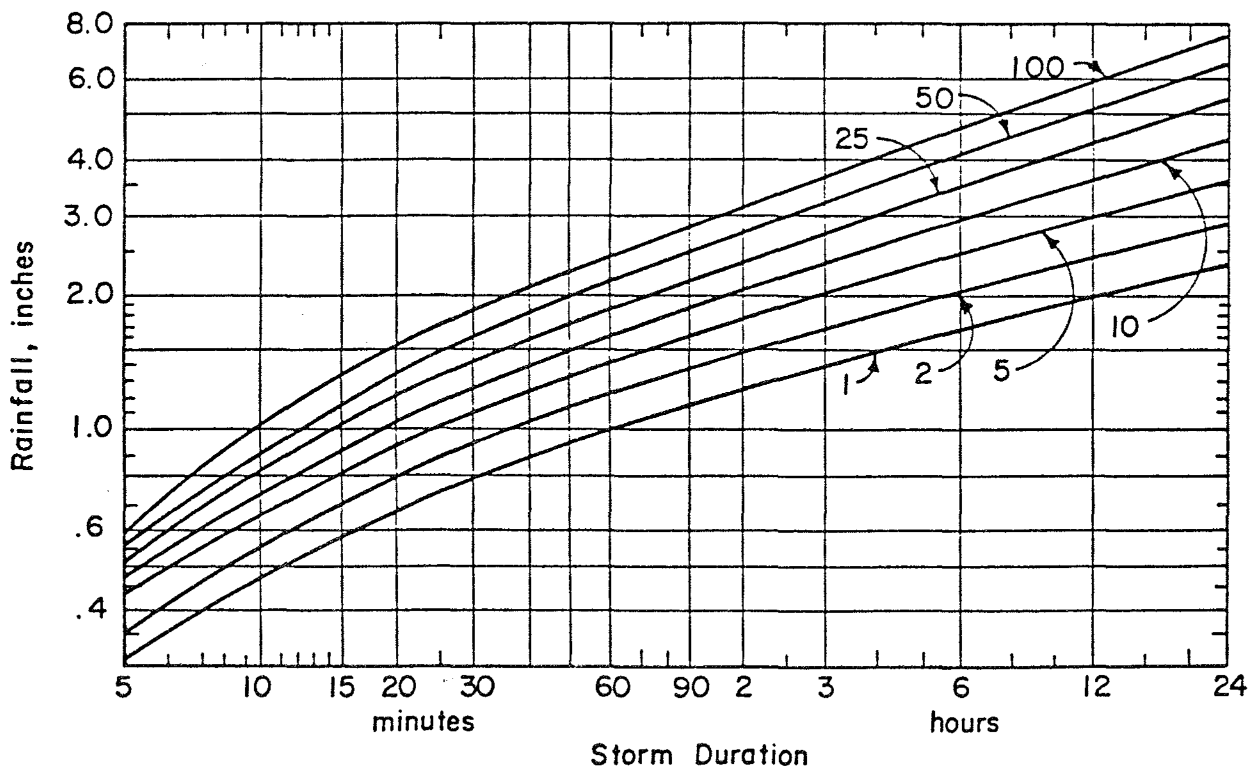


Hour/Min		P_x/P_{24}	Hour/Min		P_x/P_{24}
1	00	.0107	12	20	.6925
2	00	.0222	12	40	.7361
3	00	.0345	13	00	.7639
4	00	.0479	13	20	.7850
5	00	.0626	13	40	.8023
6	00	.0790	14	00	.8170
6	20	.0849	14	20	.8299
6	40	.0910	14	40	.8415
7	00	.0975	15	00	.8520
7	20	.1043	15	20	.8616
7	40	.1114	15	40	.8705
8	00	.1190	16	00	.8788
8	20	.1270	16	20	.8866
8	40	.1356	16	40	.8940
9	00	.1449	17	00	.9009
9	20	.1549	17	20	.9075
9	40	.1659	17	40	.9138
10	00	.1781	18	00	.9199
10	20	.1918	19	00	.9365
10	40	.2077	20	00	.9515
11	00	.2266	21	00	.9651
11	20	.2506	22	00	.9776
11	40	.2843	23	00	.9892
12	00	.3773	24	00	1.0000

*Px/P24 equals cumulative percentage rainfall as a fraction of the total 24 hour rainfall.

INTENSITY-DURATION-FREQUENCY CURVES*





*Source: Pennsylvania Dept. of Transp. Design Rainfall Curves (1986).

RUNOFF CURVE NUMBERS AND PERCENT IMPERVIOUSNESS VALUES*

Cover Description		Curve numbers for hydrologic soil group			
Land Use/Cover Type	Average percent impervious area	A	B	C	D
Open space (lawns, parks, golf courses, cemeteries, etc.):					
Good condition (grass cover greater than 75%)...		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc.(excluding right-of-way).............		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)..................		98	98	98	98
Paved; open ditches (including right-of-way).		83	89	92	93
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 (townhouses)	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		30	55	70	77
Agriculture		Refer to Table 2-2b in source document (TR55) by crop type and treatment.			

*Source: Soil Conservation Service Technical Release No. 55, Second Edition, June 1986.

RUNOFF COEFFICIENTS FOR THE RATIONAL METHOD*

	HYDROLOGIC SOIL GROUP AND SLOPE RANGE											
	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	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
Land	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

RUNOFF COEFFICIENTS FOR THE RATIONAL METHOD* (Cont'd)

	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%+
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

*Source: Rawls, W.J., S.L. Wong and R.H. McCuen, 1981. Comparison of urban flood frequency procedures. Preliminary draft report prepared for the Soil Conservation Service, Beltsville, Maryland.

[a]Runoff coefficients for storm recurrence intervals less than 25 years.

[b]Runoff coefficients for storm recurrence intervals of 25 years or more.

MANNING 'n' VALUES BY TYPICAL REACH DESCRIPTION

	Reach Description	Manning 'n'
1)	Natural stream, clean, straight, no rifts or pools	0.030
2)	Natural stream, clean, winding, some pools and shoals	0.040
3)	Natural stream, winding, pools, shoals, stony with some weeds	0.050
4)	Natural stream, sluggish with deep pools and weeds	0.070
5)	Natural stream or swale, very weedy or with timber underbrush	0.100
6)	Concrete pipe, culvert or channel	0.012
7)	Corregated metal pipe	0.012-0.027*

* Depending upon type and diameter.

PERMISSIBLE VELOCITIES FOR CHANNELS*

CHANNEL LINING	PERMISSIBLE CHANNEL VELOCITY (FEET PER SECOND)		
Vegetation			
Alfalfa	2.5	-	3.5
Bermudagrass	4	-	8
Crabgrass	2.5	-	3.5
Crownvetch	3	-	5
Kentucky Bluegrass	4	-	7
Kentucky 31 Tall Fescue	2.5	-	7
Red Clover or Red Fescue	2.5	-	3.5
Reed Canary	3	-	5
Ryegrass	2.5	-	3.5
Small Grains	2.5	-	3
Smooth Brome	3	-	7
Sudan Grass or Timothy	2.5	-	3.5
Bare Earth, Easily Eroded			
Fine Sand	1.5		
Sand Loam	1.75		
Silt Loam or Alluvial Silts, Loose	2		
Firm Loam	2.25		
Bare Earth, Erosion Resistant			
Fine Gravel	2.5		
Stiff Clay or Alluvial Silts, Firm	3		
Loam to Cobbles (graded)	3.75		
Silt to Cobbles (graded or Coarse Gravel)	4		
Cobbles and Stones or Shales and Hardpans	5		
Durable Bedrock	8		
Other			
Plastic	4		
6" Rip Rap	6		
Asphalt	7		
9" Rip Rap	8		
12" Rip Rap or Wood	9		
Concrete or Steel	12		

*These values, if applied to uniform, straight channels, may be considered in accordance with Chapter 102.12 of the Erosion Control Rules and Regulations. However, slope, soil condition, climate and management must be considered in channel design. If different channel linings exist in a channel, and size and slope do not change, design the channel for the lining with the lower velocity listed. Where velocity ranges are listed, the lower velocity is for design with easily eroded soils and slopes greater than 10%. The higher velocity is for design with erosion resistant soils and slopes less than 5%. Filtration and/or sedimentation in the channel is encouraged. However, this must be considered for velocity determination in the design of the channel cross-section.

Source: Department of Environmental Resources, Soil Erosion and Sedimentation Control Manual, Feb. 1985, Appendix 67.

APPENDIX D

CALIBRATED PSRM PEAK FLOW VALUES FOR THE MONOCACY CREEK WATERSHED

CALIBRATED PSRM PEAK FLOW VALUES FOR THE MONOCACY CREEK WATERSHED

SUBAREA NO.	2 YEAR SUBAREA PEAK	2 YEAR TOTAL PEAK	10 YEAR SUBAREA PEAK	10 YEAR TOTAL PEAK	25 YEAR SUBAREA PEAK	25 YEAR TOTAL PEAK	50 YEAR SUBAREA PEAK	50 YEAR TOTAL PEAK	100 YEAR SUBAREA PEAK	100 YEAR TOTAL PEAK
WEST BRANCH										
1	12.8(cfs)	12.8	31.6(cfs)	31.6	52.1(cfs)	52.1	68.6(cfs)	68.6	96.5(cfs)	96.5
2	89.9	101.2	251.7	279.1	446.9	487.3	607.3	661.0	884.0	959.0
3	40.5	139.3	127.1	405.5	223.1	687.3	304.0	910.2	461.9	1282.6
4	17.8	17.8	51.1	51.1	92.7	92.7	129.0	129.0	194.8	194.8
5	41.7	186.5	129.2	545.3	231.4	926.4	910.2	1232.7	451.2	1751.7
6	45.8	45.8	130.1	130.1	231.4	231.4	316.3	316.3	465.5	465.5
7	49.8	93.6	144.9	271.2	258.0	462.6	352.3	615.3	516.9	874.0
8	71.7	71.7	218.5	218.5	386.4	386.4	540.3	540.3	819.9	819.9
9	5.4	167.0	21.8	502.3	44.9	875.1	65.8	1175.6	103.9	1689.2
10	18.9	18.9	28.8	28.8	36.8	36.8	42.4	42.4	51.3	51.3
11	3.4	178.0	21.3	525.2	45.0	913.2	68.0	1229.1	110.7	1773.8
12	21.9	21.9	68.2	68.2	125.2	125.2	174.0	174.0	260.2	260.2
13	1.4	189.6	6.7	579.1	22.4	1020.9	41.9	1386.7	83.1	2021.1
14	80.8	263.5	239.3	763.4	423.7	1300.8	575.8	1731.4	838.4	2459.8
15	2.3	449.3	5.2	1281.4	7.7	2147.3	9.4	2835.0	12.1	4098.0
16	59.1	497.1	158.3	1399.5	272.3	2348.0	368.0	3101.6	537.0	4372.7
17	47.2	47.2	132.6	132.6	230.5	230.5	311.8	311.8	453.4	453.4
18	65.8	569.5	140.2	1579.0	220.9	2635.8	286.7	3473.6	399.4	4884.4
19	73.3	579.4	130.0	1598.9	199.6	2694.4	265.6	3572.8	397.9	5056.5
20	28.0	448.6	96.6	1433.5	193.4	2618.8	284.3	3546.0	459.9	5132.3
21	39.2	472.7	128.0	1520.7	239.0	2718.1	336.3	3622.9	513.9	5197.5
22	10.5	327.4	34.4	1463.1	65.7	2611.9	93.4	3557.2	144.7	5111.5
23	6.5	326.1	11.2	1451.0	17.8	2609.2	24.4	3530.2	37.5	4909.4
EAST BRANCH										
24	19.1(cfs)	19.1	54.1(cfs)	54.1	96.1(cfs)	96.1	131.0(cfs)	131.0	191.1(cfs)	191.1
25	17.2	17.2	49.2	49.2	86.6	86.6	116.9	116.9	168.5	168.5
26	43.6	77.1	124.3	204.0	205.1	349.7	266.6	465.4	366.4	659.9
27	21.5	21.5	41.4	41.4	61.8	61.8	78.1	78.1	105.8	105.8
28	106.2	185.5	233.7	452.8	373.6	743.2	488.4	975.4	687.1	1370.6
29	23.9	23.9	60.1	60.1	101.7	101.7	136.5	136.5	198.0	198.0
30	54.9	250.4	174.6	659.3	303.5	1094.2	425.9	1437.7	644.2	2012.8
31	27.1	27.1	70.7	70.7	120.2	120.2	161.3	161.3	233.5	233.5
32	9.7	282.6	28.2	738.4	45.0	1219.9	57.2	1600.1	76.4	2237.2
33	11.6	11.6	31.1	31.1	53.4	53.4	71.9	71.9	103.6	103.6
34	27.0	317.5	84.8	832.8	150.3	1371.1	202.6	1794.5	290.0	2501.8
35	11.3	11.3	30.1	30.1	49.6	49.6	64.9	64.9	90.3	90.3
36	40.3	360.4	111.3	934.3	184.5	1525.7	241.3	1988.9	334.4	2761.1
37	38.2	386.7	114.0	994.2	195.4	1618.9	258.9	2111.0	363.3	2936.2
38	25.6	25.6	70.0	70.0	120.3	120.3	160.3	160.3	227.7	227.7
39	43.2	442.5	115.0	1130.3	194.5	1834.2	257.3	2389.0	362.4	3319.9
40	36.7	36.7	100.4	100.4	167.3	167.3	219.9	219.9	307.4	307.4
41	5.1	476.5	11.4	1210.8	16.6	1957.6	20.3	2543.9	26.1	3524.3
42	28.2	188.6	77.3	792.4	134.5	1505.8	181.3	2068.7	261.7	3185.7
43	78.1	197.4	179.9	881.1	304.0	1613.6	413.8	2230.5	617.6	3194.6
44	42.1	42.1	116.7	116.7	196.6	196.6	259.7	259.7	364.5	364.5
45	33.4	68.4	99.5	178.2	179.3	181.2	247.7	250.0	373.2	374.1
46	25.9	199.2	81.7	940.8	149.4	1709.5	209.9	2393.6	319.5	3307.0
47	16.8	154.5	53.2	858.4	93.8	1638.0	131.4	2341.9	198.1	3165.5
48	69.6	69.6	169.0	169.0	285.0	285.0	384.7	384.7	565.4	565.4
49	40.7	40.7	108.1	108.1	186.2	186.2	250.7	250.7	362.5	362.5

SUBAREA NO.	2 YEAR SUBAREA PEAK	2 YEAR TOTAL PEAK	10 YEAR SUBAREA PEAK	10 YEAR TOTAL PEAK	25 YEAR SUBAREA PEAK	25 YEAR TOTAL PEAK	50 YEAR SUBAREA PEAK	50 YEAR TOTAL PEAK	100 YEAR SUBAREA PEAK	100 YEAR TOTAL PEAK
EAST BRANCH										
50	63.0	156.8	189.1	392.8	320.2	636.5	421.3	830.1	586.3	1162.4
51	15.5	15.5	44.7	44.7	79.2	79.2	108.3	108.3	159.5	159.5
52	25.9	185.8	85.0	476.3	145.7	777.6	191.8	1016.7	266.1	1415.3
53	48.1	48.1	134.1	134.1	235.8	235.8	319.4	319.4	463.8	463.8
54	16.0	241.5	52.3	629.7	89.5	1029.7	117.9	1344.8	163.5	1866.3
55	51.5	51.5	149.2	149.2	264.6	264.6	362.5	362.5	532.1	532.1
56	67.7	350.6	196.4	929.0	353.0	1536.8	482.0	2014.6	704.9	2802.4
57	25.2	21.8	42.9	415.7	57.7	849.0	68.7	1284.9	89.6	2144.4
58	57.3	68.3	171.8	368.6	311.6	862.5	439.1	1334.1	675.5	2019.3
59	8.8	222.3	26.0	1066.4	44.0	2285.3	58.0	3477.9	81.0	4986.2
60	35.8	35.8	99.4	99.4	176.4	176.4	243.8	243.8	368.3	368.3
61	71.2	283.0	157.0	1151.7	260.0	2363.0	350.7	3450.0	518.3	5033.5
62	50.1	315.7	140.1	1169.9	252.0	2354.7	350.3	3456.8	530.8	5000.3
63	6.2	315.4	19.1	1147.5	32.3	2336.1	42.5	3453.4	58.9	4879.8
MAIN BRANCH										
64	23.7(cfs)	440.9	75.7(cfs)	1955.4	137.9(cfs)	3026.7	194.3(cfs)	4410.4	297.2(cfs)	5734.2
65	34.0	34.0	77.6	77.6	130.1	130.1	177.5	177.5	266.5	266.5
66	.8	450.1	1.2	1961.7	1.7	3023.3	2.6	4378.9	5.3	5679.1
67	61.0	61.0	202.1	202.1	379.8	379.8	535.7	535.7	820.6	820.6
68	100.0	115.3	213.1	331.8	341.4	600.6	452.5	830.6	657.5	1240.5
69	8.7	510.2	28.5	2026.5	60.7	3087.9	92.1	4379.8	154.1	5653.9
70	42.9	512.2	105.7	2023.4	181.8	3044.1	246.9	4301.9	362.7	5580.1
71	35.9	508.3	96.8	2018.4	173.3	3018.0	240.1	4287.5	361.5	5561.1
72	99.0	99.0	178.5	178.5	258.4	258.4	324.9	324.9	446.1	446.1
73	25.2	522.7	53.4	2021.2	89.6	3007.6	122.4	4265.2	183.9	5514.6
74	38.5	525.1	83.6	2021.1	136.7	2952.7	183.0	4175.6	267.9	5467.3
75	9.4	9.4	33.6	33.6	63.3	63.3	88.8	88.8	134.4	134.4
76	70.9	526.9	153.9	2018.2	260.2	2942.5	358.0	4162.3	545.6	5411.8
77	12.9	12.9	46.5	46.5	86.8	86.8	122.7	122.7	187.9	187.9
78	5.0	532.3	20.6	2023.6	39.6	2915.0	55.4	4121.8	82.2	5409.7
79	84.8	539.0	153.4	2021.9	223.7	2914.9	282.8	4109.4	391.1	5363.4
80	55.0	535.5	115.5	2016.2	190.1	2884.1	256.3	4053.8	378.0	5341.0
81	49.4	49.4	94.1	94.1	140.1	140.1	178.6	178.6	248.8	248.8
82	22.1	22.1	66.5	66.5	121.9	121.9	170.9	170.9	261.5	261.5
83	4.6	4.6	18.3	18.3	36.7	36.7	53.5	53.5	85.2	85.2
84	42.7	86.7	115.1	225.9	210.6	390.5	297.3	529.8	460.3	777.7
85	56.7	101.2	110.0	265.6	168.4	452.1	217.6	608.5	305.8	884.3
86	86.3	567.8	162.2	2035.4	247.9	2898.1	321.5	4063.5	454.8	5309.1
87	32.2	32.2	60.4	60.4	89.1	89.1	113.2	113.2	156.9	156.9
88	82.8	573.0	155.6	2023.7	238.3	2853.7	310.2	4025.9	442.1	5286.5
89	28.4	587.6	78.2	2032.7	133.5	2861.1	179.1	4001.5	259.0	5275.1
90	103.5	590.4	189.7	2033.5	284.7	2860.3	367.8	3988.7	524.6	5259.2
91	32.8	589.3	62.3	2032.5	99.8	2855.0	132.2	3983.1	190.0	5229.9
92	113.1	590.5	206.8	2022.0	320.1	2828.5	415.7	3968.6	584.0	5216.6
93	63.7	63.7	121.6	121.6	181.8	181.8	232.9	232.9	326.9	326.9
94	309.4	337.8	513.5	568.5	695.5	775.8	835.0	935.9	1071.3	1209.5
95	98.8	98.8	158.6	158.6	214.7	214.7	258.7	258.7	334.2	334.2
96	85.7	504.2	138.2	837.6	185.7	1135.3	222.3	1364.7	284.3	1755.9
97	373.2	840.2	606.4	1387.3	815.9	1866.8	977.0	2233.2	1250.1	2854.9
98	322.9	1344.0	513.4	2126.0	690.3	2967.3	828.0	3946.6	1062.8	5189.0
99	146.7	1365.2	232.0	2172.9	311.2	2990.2	372.1	3945.9	474.2	5186.0
100	394.7	1365.8	631.5	2217.7	858.8	3128.5	1036.4	3927.1	1337.3	5136.2
101	37.1	1349.0	60.8	2218.9	84.5	3112.3	103.6	3924.8	137.1	5132.0

CHAPTER X. PRIORITIES FOR IMPLEMENTATION OF THE PLAN

The Monocacy Creek Storm Water Management Plan preparation process is complete with the Northampton County and Lehigh County adoption of the draft Plan and submission of the final Plan to DER for approval. Procedures for the review and adoption of the Plan are included in Chapter XI. Subsequent activities to carry out the provisions of the Plan are considered by DER to be part of the implementation of the Plan. The initial step of Plan implementation is DER approval. Plan approval sets in motion the mandatory schedule of adoption of municipal ordinance provisions to implement the storm water management criteria. Monocacy Creek municipalities would have six months from DER approval within which to adopt the necessary ordinance provisions. Failure to do so could result in the withholding of all state funds to the municipality(ies) per Act 167.

Additional implementation activities are the formal publishing of the final Plan after DER approval, development of a local program to coordinate with DER regarding permit reviews for stream encroachments, diversions, etc., and development of a systematic approach for correction of existing storm drainage problem areas. The priorities for plan implementation are presented in detail below in (essentially) chronological order.

A. DER Approval of the Plan

Upon adoption of the watershed plan by Northampton (and Lehigh) County, the Plan is submitted to DER for approval. The DER review process involves determination that all of the activities specified in the approved Scope of Study have been satisfactorily completed in the Plan. Further, the Department will only approve the Plan if it determines the following:

1. That the Plan is consistent with municipal floodplain management plans, State programs which regulate dams, encroachments and other water obstructions, and State and Federal flood control programs; and

2. That the Plan is compatible with other watershed storm water plans for the basin in which the watershed is located and is consistent with the policies and purposes of Act 167.

DER action to either approve or disapprove the Plan must take place within ninety (90) days of receipt of the Plan by the Department. Otherwise, the Plan would be approved by default.

B. Publishing the Plan

Consistent with the Monocacy Creek Scope of Study, the JPC will publish additional copies of the Watershed Plan after DER approval. At minimum, two copies of the Plan will be provided to each municipality. Additional separate copies of the Monocacy Creek Act 167 Storm Water Management Ordinance will be published for use by the municipalities.

C. Development of a Local Program to Coordinate with DER Regarding Chapter 105 and Chapter 106 Permit Application Reviews

Stream encroachments, stream enclosures, waterway diversions, water obstructions and other activities regulated by Chapter 105 and Chapter 106 of DER's Rules and Regulations may have a bearing on the effectiveness of the runoff control strategy developed for the Monocacy Creek Basin. Activities of this type may modify the conveyance characteristics of the watershed and, hence, impact on 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 DER permitting process is consistent with the adopted and approved watershed plan, a local review of Chapter 105 and Chapter 106 permit applications should be coordinated with the DER review process.

The local review for Northampton and Lehigh Counties would be performed by the JPC and would be accomplished through monitoring of the applications as published in the Pennsylvania Bulletin. The JPC would be responsible for providing comments consistent with the adopted Act 167 plan within the stated DER review period. Further, the JPC would keep records of applications reviewed and the DER action.

D. Municipal Adoption of Ordinance Provisions to Implement the Plan

The key ingredient for implementation of the Storm Water Management Plan is the adoption of the necessary ordinance provisions by the Monocacy Creek municipalities. Provided as part of the Plan is the Monocacy Creek Watershed Act 167 Storm Water Management Ordinance which is a single purpose storm water ordinance that could be adopted by each municipality essentially as is to implement the Plan. The single purpose ordinance was chosen for ease of incorporation into the existing structure of municipal ordinances. All that would be required of any municipality would be to adopt the ordinance itself and adopt the necessary tying provisions into the existing subdivision and land development ordinance and zoning ordinance. The tying provisions would simply refer any applicable regulated activities within the Monocacy Creek Watershed to the single purpose ordinance from the other ordinances.

It is not required, however, that a municipality adopt the single purpose ordinance. At the municipality's discretion, it may opt to incorporate all of the necessary provisions into the existing ordinances rather than adopt a separate ordinance. In this event, the municipalities must ensure that the amended ordinances satisfactorily implement the approved Plan.

E. Development of a Systematic Approach for Correction of Existing Storm Drainage Problem Areas

Correction of the existing storm drainage problem areas in the watershed is not specifically part of the Act 167 planning process. However, the development of the watershed plan has provided a framework for their correction for the following reasons: (1) existing storm drainage problems have been documented through interaction with the Watershed Advisory Committee; (2) implementation of the runoff control criteria specified in the Plan will prevent the existing drainage problems from becoming worse (and prevent the creation of new drainage problem areas); and (3) the hydrologic model developed to formulate the runoff control criteria could be used as an analytical tool for designing engineering solutions to existing drainage problems.

With the above in mind, each municipality within the Monocacy Creek Basin should take the following steps to implement solutions to the existing storm drainage problem areas:

1. Prioritize the list of storm drainage problems within the municipality based on frequency of occurrence, potential for injury to persons or property, damage history, public perception of the problems, and other appropriate criteria.

2. For the top priority drainage problems in the municipality, conduct detailed engineering evaluations to determine the exact nature of the problems (if not known), determine alternative solutions, provide cost estimates for the alternative solutions, and recommend a course of municipal action. The number of drainage problems to be evaluated by a municipality as a first cut from the priority list should be based on a schedule commensurate with completing engineering studies on all problem areas within approximately five years. The Monocacy Creek hydrologic model would be available at the JPC office to provide flow data as input to the engineering studies.

3. On the priority and cost basis, incorporate implementation of recommended solutions to the drainage problems in the annual municipal capital budget or the municipal maintenance budget as funds are available.

> The number of drainage problems corrected in a given year should be based on a maximum ten-year schedule of resolving all existing documented drainage problems in the municipality for which cost-effective solutions exist.

The above-stated procedure for dealing with existing storm drainage problem areas is not a mandatory action placed on municipalities with the adoption of the watershed plan. Rather, it represents one systematic method to approach the problems uniformly throughout the watershed and attempt to improve the current runoff situation in the basin. The key elements involved in the success of the remedial strategy will be the dedication of the municipalities to construct the corrective measures and the consistent and proper application of the runoff control criteria specified in the Plan. The latter element is essential to ensure that remedial measures do not become obsolete (under-designed) by increases in peak flows with development.

CHAPTER XI. PLAN REVIEW, ADOPTION AND UPDATING PROCEDURES

A. Plan Review and Adoption

The opportunity for local review of the draft Storm Water Management Plan is a prerequisite to county adoption of the Plan. Local review of the Plan is composed of three parts, namely Watershed Advisory Committee review, municipal review and County Council review. Local review of the draft Plan is initiated with the completion of the Plan by the JPC and distribution to the Watershed Advisory Committee. Presented below is a chronological listing and brief narrative of the required local review steps through County adoption.

1. Watershed Advisory Committee Review - This body has been formed to assist in the development of the Monocacy Creek Watershed Plan. Municipal members of the Committee have provided input data to the process in the form of storm drainage problem area documentation, storm sewer documentation, proposed solutions to drainage problems, etc.. The Committee met on six occasions to review the progress of the Plan. Municipal representatives on the Committee have the responsibility to report on the progress of the Plan to their respective municipalities. Review of the draft Plan by the Advisory Committee will be expedited by the fact that the members are already familiar with the objectives of the Plan, the runoff control strategy employed and the basic contents of the Plan. The output of the Watershed Advisory Committee review would be a letter outlining the Committee comments which would be included with the draft Plan for municipal and County consideration.

2. Municipal Review - Act 167 specifies that prior to adoption of the draft Plan by the County, the planning commission and governing body of each municipality in the watershed must review the Plan for consistency with other plans and programs affecting the watershed. Of primary concern during the municipal review would be the draft "Monocacy Creek Watershed Act 167 Storm Water Management Ordinance" which would implement the Plan through municipal adoption. The output of the municipal review would be a letter directed to the County Council outlining the municipal suggestions, if any, for revising the draft Plan (or Ordinance) prior to County adoption.

3. County Review and Adoption - Upon completion of the review by the Watershed Advisory Committee and each municipality, the draft Plan will be submitted to both the Northampton County Council and Lehigh County Board of Commissioners for their consideration.

The County review of the draft Plan will include a detailed review by the Council and Board of Commissioners and an opportunity for public input through the holding of a public hearing. A public hearing on the draft Plan must be held with a minimum two-week notice period with copies of the draft Plan available for inspection by the general public. Any modifications to the draft Plan would be made by the Counties based upon input from the public hearing, comments received from the municipalities in the watershed, or their own review. Adoption of the draft Plan by Northampton County and Lehigh County would be by resolution and require an affirmative vote of the majority of members of the Council and Board of Commissioners, respectively.

The adopted Plan would be submitted by Northampton County to DER for their consideration for approval. Accompanying the adopted Plan to DER would be the review comments of the municipalities.

B. Procedure for Updating the Plan

Act 167 specifies that the County must review and, if necessary, revise the adopted and approved watershed plan every five years, at minimum. Any proposed revisions to the Plan would require municipal and public review prior to County adoption consistent with the procedures outlined above. An important aspect of the plan is a procedure to monitor the implementation of the Plan and initiate review and revisions in a timely manner. The process to be used for the Monocacy Creek Storm Water Management Plan will be as outlined below.

1. Monitoring of the Plan Implementation - The Joint Planning Commission will be responsible for monitoring the implementation of the Plan by maintaining a record of all development activities within the watershed. Development activities are defined as those activities regulated by the Storm Water Management Plan as described in Chapter IX and included in the recommended municipal ordinance. Specifically, the JPC will monitor the following data records:

    (a) All subdivision and land developments subject to review per the Plan which have been approved within the watershed.

    (b) All building permits subject to review per the Plan which have been approved within the watershed.

    (c) All DER permits issued under Chapter 105 (Dams and Waterway Management) and Chapter 106 (Floodplain

Management) including location and design capacity (if applicable).

2. Review of Adequacy of Plan - The Watershed Advisory Committee will be convened annually to review the Storm Water Management Plan and determine if the Plan is adequate for minimizing the runoff impacts of new development. At minimum, the information to be reviewed by the Committee will be as follows:

   (a) Development activity data as monitored by the JPC.

   (b) Information regarding additional storm drainage problem areas as provided by the municipal representatives to the Watershed Advisory Committee.

   (c) Zoning amendments within the watershed.

   (d) Impacts associated with any regional or sub-regional detention alternatives implemented within the watershed.

   (e) Adequacy of the administrative aspects of regulated activity review.

   The Committee will review the above data and make recommendations to the County as to the need for revisions to the Monocacy Creek Watershed Storm Water Management Plan. Northampton and Lehigh County will review the recommendations of the Watershed Advisory Committee and determine if revisions are to be made. A revised Plan would be subject to the same rules of adoption as the original Plan preparation. Should the County(ies) determine that no revisions to the Plan are required for a period of five consecutive years, the County(ies) will adopt a resolution stating that the Plan has been reviewed and been found satisfactory to meet the requirements of Act 167 and forward the resolution to DER.

RELEASE RATE SUMMARY TABLE

SUBAREA	RELEASE RATE (%)	SUBAREA	RELEASE RATE (%)	SUBAREA	RELEASE RATE (%)
1	90	35	80	69	30/100
2	90	36	80	70	30/100
3	90	37	70	71	30/100
4	80	38	80	72	30/100
5	70	39	70	73	See Map
6	100	40	70	74	See Map
7	80	41	70	75	30/100
8	90	42	70	76	See Map
9	80	43	50	77	30/100
10	80	44	80	78	No Detention*
11	80	45	50	79	No Detention*
12	90	46	50	80	See Map
13	70	47	50	81	30/100
14	70	48	100	82	30/100
15	70	49	90	83	30/100
16	70	50	70	84	30/100
17	80	51	80	85	30/100
18	50	52	70	86	See Map
19	50	53	80	87	No Detention*
20	50	54	60	88	No Detention*
21	50	55	80	89	No Detention*
22	50	56	60	90	No Detention*
23	30/100	57	50	91	No Detention*
24	100	58	50	92	No Detention*
25	100	59	50	93	100
26	90	60	50	94	100
27	90	61	50	95	90
28	90	62	50	96	90
29	80	63	30/100	97	80
30	80	64	30/100	98	No Detention*
31	80	65	30/100	99	No Detention*
32	80	66	30/100	100	No Detention*
33	80	67	30/100	101	No Detention*
34	80	68	30/100		

*No detention controls are necessary for these watershed areas provided that adequate downstream capacity can be shown for increased peak flows. (See Plan for additional details.)

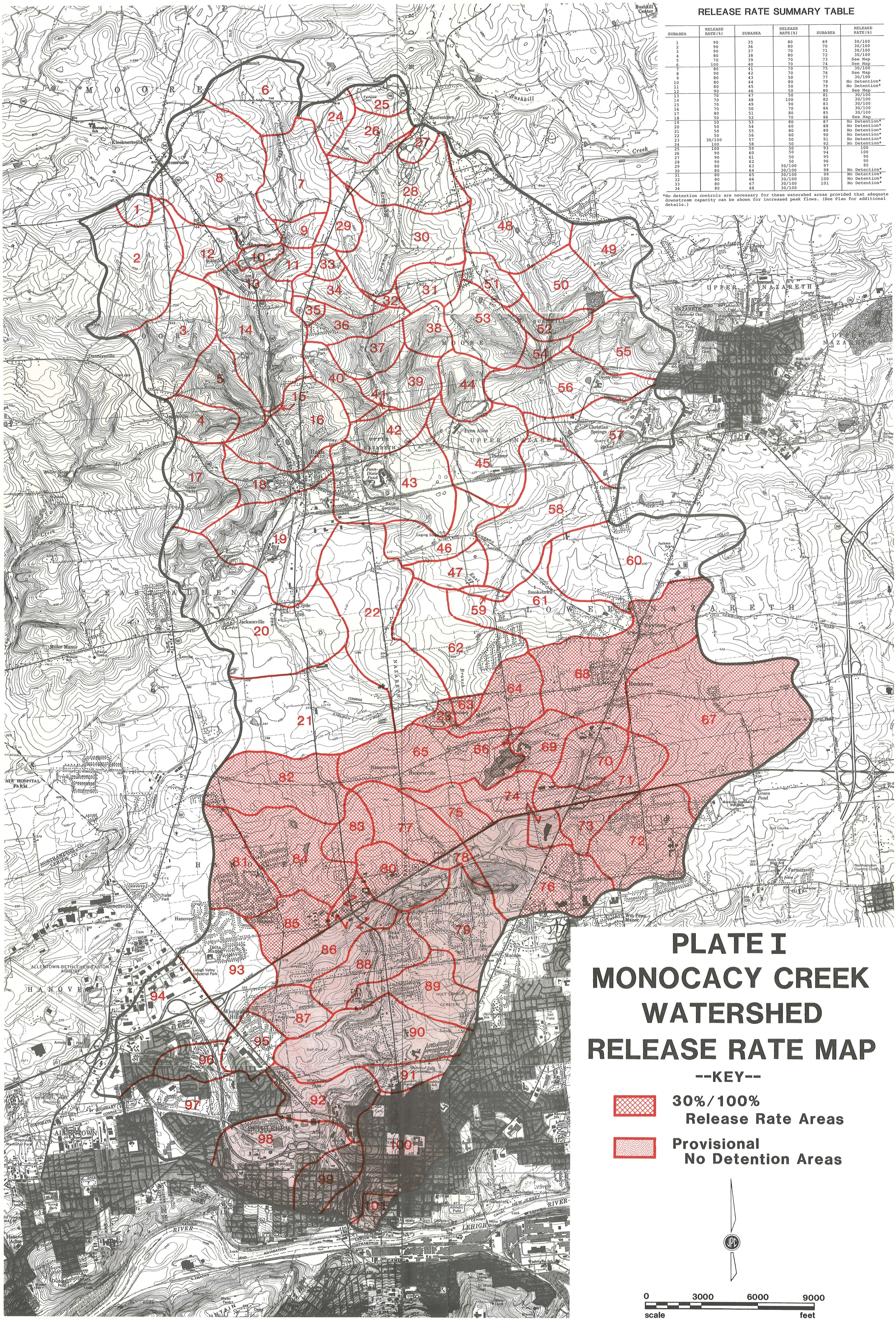


PLATE I
MONOCACY CREEK WATERSHED RELEASE RATE MAP

--KEY--

30%/100% Release Rate Areas

Provisional No Detention Areas

0 3000 6000 9000
scale feet