State of New Jersey
Department of Transportation

Drainage Design Manual

August 2006

1.0 General Information

1.1 Introduction

Investigation of the impacts of surface water on the highway roadway, channels, and surrounding land is an integral part of every highway design. The end product of this investigation is a design, included in the plans, that provides an economical means of accommodating surface water to minimize adverse impacts in accordance with the design procedures.

Traffic safety is intimately related to surface drainage. Rapid removal of stormwater from the pavement minimizes the conditions which can result in the hazardous phenomenon of hydroplaning. Adequate cross-slope and longitudinal grade enhance such rapid removal. Where curb and gutter are necessary, the provision of sufficient inlets in conjunction with satisfactory cross-slope and longitudinal slope are necessary to efficiently remove the water and limit the spread of water on the pavement. Inlets at strategic points on ramp intersections and approaches to superelevated curves will reduce the likelihood of gutter flows spilling across roadways. Satisfactory cross-drainage facilities will limit the buildup of ponding against the upstream side of roadway embankments and avoid overtopping of the roadway.

Stormwater management is an increasingly important consideration in the design of roadway drainage systems. Existing downstream conveyance constraints, particularly in cases where the roadway drainage system connects to existing pipe systems, may warrant installation of detention/recharge basins to limit the peak discharge to the capacity of the downstream system. Specific stormwater management requirements to control the rate and volume of runoff may be dictated by various regulatory agencies.

Water quality is also an increasingly important consideration in the design of roadway drainage systems, particularly as control of non-point source pollution is implemented. Specific water quality requirements may be dictated by various regulatory agencies.

Detailed requirements regarding water quality control are included in Section 12.0 of this Manual and the separate document prepared by the New Jersey Department of Environmental Protection (NJDEP) entitled Stormwater Best Management Practices Manual.

The optimum roadway drainage design should achieve a balance among public safety, the capital costs, operation and maintenance costs, public convenience, environmental enhancement and other design objectives.

The purpose of this manual is to provide the technical information and procedures required for the design of culverts, storm drains, channels, and stormwater management facilities. This section contains design criteria and information that will be required for the design of highway drainage structures. The complexity of the subject requires referring to additional design manuals and reports for more detailed information on several subjects.

1.2 Definitions and Abbreviations

Following is a list of important terms which will be used throughout this volume.

AWS - Allowable water surface elevations - The water surface elevation above which damage will occur.

AHW - Allowable headwater elevation - The allowable water surface elevation upstream from a culvert.

Backwater - The increased depth of water upstream from a dam, culvert, or other drainage structure due to the existence of such obstruction.

Best Management Practice (BMP) – A structural feature or non-structural development strategy designed to minimize or mitigate for impacts associated with stormwater runoff, including flooding, water pollution, erosion and sedimentation, and reduction in groundwater recharge.

Bioretention – A water quality treatment system consisting of a soil bed planted with native vegetation located above an underdrained sand layer. It can be configured as either a bioretention basin or a bioretention swale. Stormwater runoff entering the bioretention system is filtered first through the vegetation and then the sand/soil mixture before being conveyed downstream by the underdrain system.

Category One Waters – Those waters designated in the tables in N.J.A.C. 7:9B-4.15(c) through (h) for the purposes of implementing the Antidegradation Polices in N.J.A.C. 7:9B-4. These waters received special protection under the Surface Water Quality Standards because of their clarity, color, scenic setting or other characteristics of aesthetic value, exceptional ecological significance, exceptional recreational significance, exceptional water supply significance or exceptional fisheries resource(s). More information on Category One Waters can be found on the New Jersey Department of Environmental Protection's (NJDEP) web sites http://www.state.nj.us/dep and http://www.state.nj.us/dep/antisprawl/c1.html.

Channel - A perceptible natural or artificial waterway which periodically or continuously contains moving water. It has a definite bed and banks which confine the water. A roadside ditch, therefore, would be considered a channel.

Culvert – A hydraulic structure that is typically used to convey surface waters through embankments. A culvert is typically designed to take advantage of submergence at the inlet to increase hydraulic capacity. It is a structure, as distinguished from a bridge, which is usually covered with embankment and is composed of structural material around the entire perimeter, although some are supported on spread footings with the stream bed serving as the bottom of the culvert. Culverts are further differentiated from bridges as having spans typically less than 25 feet.

Dam - Any artificial dike, levy or other barrier together with appurtenant works, which impounds water on a permanent or temporary basis, that raises the water level 5 feet or more above its usual mean low water height when measured from the downstream toe-of-dam to the emergency spillway crest or, in the absence of an emergency spillway, to the top of dam.

Design Flow - The flow rate at a selected recurrence interval.

Fluvial Flood - A flood which is caused entirely by runoff from rainfall in the upstream drainage area and is not influenced by the tide or tidal surge.

Floodplain - The area described by the perimeter of the Design Flood. That portion of a river valley which has been covered with water when the river overflowed its banks at flood stage. An area designated by a governmental agency as a floodplain.

Pipe - A conduit that conveys stormwater which is intercepted by the inlets, to an outfall where the stormwater is discharged to the receiving waters. The drainage system consists of differing lengths and sizes of pipe connected by drainage structures.

Recurrence Interval - The average interval between floods of a given magnitude.

Regulatory Flood – For delineated streams (i.e., those for which a State Adopted Flood Study exists), it is the Flood Hazard Area Design Flood, which is the 100-year peak discharge increased by 25 percent. State Adopted Flood Studies can be obtained from the NJDEP Bureau of Floodplain Management. For non-delineated streams, it is the 100-year peak discharge, based on fully developed conditions within the watershed.

Scour – Erosion of stream bed or bank material due to flowing water; often considered as being localized.

Stream Encroachment - Any manmade alteration, construction, development, or other activity within a floodplain.

Time of Concentration (T_c) – Time required for water to flow from the most hydraulically distant (but hydraulically significant) point of a watershed, to the outlet.

Total Suspended Solids (TSS) - Solids in water that can be trapped by a filter, which include a wide variety of material, such as silt, decaying plant and animal matter, industrial wastes, and sewage.

1.3 Design Procedure Overview

This chapter outlines the general process of design for roadway drainage systems. Detailed information regarding drainage design is included in the remainder of this Manual.

A. Preliminary investigation will be performed using available record data, including reports, studies, plans, topographic maps, etc., supplemented with field reconnaissance. Information should be obtained for the project area and for adjacent stormwater management projects that may affect the highway drainage.

B. **Site Analysis**: At each site where a drainage structure(s) will be constructed, the following items should be evaluated as appropriate from information given by the preliminary investigation:

   1. Drainage Area.
   2. Land Use.
   3. Allowable Headwater.
   4. Effects of Adjacent Structures (upstream and downstream).
   5. Existing Streams and Discharge Points.
   6. Stream Slope and Alignment.
   7. Stream Capacity.
   8. Soil Erodibility.
   9. Environmental permit concerns and constraints.

   Coordination with representatives of the various environmental disciplines is encouraged.

C. **Recurrence Interval**: Select a recurrence interval in accordance with the design policy set forth in Section 2.0.

D. **Hydrologic Analysis**: Compute the design flow utilizing the appropriate hydrologic method outlined in Section 3.0.

E. **Hydraulic Analysis**: Select a drainage system to accommodate the design flow utilizing the procedures outlined in the following parts:

1. Channel Design – Section 4.0
2. Drainage of Highway Pavements – Section 5.0
3. Storm Drains - Section 6.0
4. Median Drainage – Section 7.0
5. Culverts - Section 8.0

F. **Environmental Considerations**: Environmental impact of the proposed drainage system and appropriate methods to avoid or mitigate adverse impacts should be evaluated. Items to be considered include:

1. Stormwater Management
2. Water Quality
3. Soil Erosion and Sediment Control
4. Special Stormwater Collection Procedures
5. Special Stormwater Disposal Procedures

These elements should be considered during the design process and incorporated into the design as it progresses.

G. **Drainage Review:** The design engineer should inspect the drainage system sites to check topography and the validity of the design. Items to check include:

1. Drainage Area
   a. Size
   b. Land Use
   c. Improvements
2. Effects of Allowable Computed Headwater
3. Performance of Existing or Adjacent Structures
   a. Erosion
   b. Evidence of High Water
4. Channel Condition
   a. Erosion
   b. Vegetation
   c. Alignment of Proposed Facilities with Channel
5. Impacts on Environmentally Sensitive Areas

2.0 Drainage Policy

2.1 Introduction

This part contains procedures and criteria that are essential for roadway drainage design.

2.2 Stormwater Management and Non-Point Source Pollution Control

Stormwater is a component of the total water resources of an area and should not be casually discarded but rather, where feasible, should be used to replenish that resource. In many instances, stormwater problems signal either misuse of a resource or unwise land activity.

Poor management of stormwater increases total flow, flow rate, flow velocity and depth of water in downstream channels. In addition to stormwater peak discharge and volume impacts, roadway construction or modification usually increases non-point source pollution primarily due to the increased impervious area. Properly designed stormwater management facilities, particularly detention/recharge basins, can also be used to mitigate non-point source pollution impacts by providing extended containment duration, thereby allowing settlement of suspended solids. Sections 2.6, 11.0 and 12.0 of this Manual and the Stormwater Best Management Practices Manual prepared by the New Jersey Department of Environmental Protection (NJDEP) provide the guidance in the planning and design of these facilities.

An assessment of the impacts the project will have on existing peak flows and watercourses shall be made by the design engineer during the initial phase. The assessment shall identify the need for stormwater management and non-point source pollution control (SWM & NPSPC) facilities and potential locations for these facilities. Mitigating measures can include, but are not limited to, detention/recharge basins, grassed swales, channel stabilization measures, and easements.

Stormwater management, whether structural or non-structural, on or off site, must fit into the natural environment, and be functional, safe, and aesthetically acceptable. Several alternatives to manage stormwater and provide water quality may be possible for any location. Careful design and planning by the engineer, hydrologist, biologist, environmentalist, and landscape architect can produce optimum results.

Design of SWM & NPSPC measures must consider both the natural and man-made existing surroundings. The design engineer should be guided by this and include measures in design plans that are compatible with the site specific surroundings. Revegetation with native, non-invasive grasses, shrubs and possibly trees may be required to achieve compatibility with the surrounding environment. Design of major SWM & NPSPC facilities may require coordination with the New Jersey Department of Transportation (NJDOT)-Landscape and Urban Design Unit, Bureau of Environmental Services, and other state and various regulatory agencies.

SWM & NPSPC facilities shall be designed in accordance with Sections 11.0 and 12.0 and the Stormwater Best Management Practices Manual prepared by the NJDEP or other criteria where applicable, as directed by the Department.

Disposal of roadway runoff to available waterways that either cross the roadway or are adjacent to it spaced at large distances, requires installation of long conveyance systems. Vertical design constraints may make it impossible to drain a pipe or swale system to existing waterways. Discharging the runoff to the groundwater with a series of leaching or seepage basins (sometimes called a Dry Well) may be an appropriate alternative if groundwater levels and non-contaminated, permeable soil conditions allow a properly designed system to function as designed. The decision to select a seepage facility design must consider geotechnical, maintenance, and possibly right-of-way (ROW) impacts and will only be allowed if no alternative exists.

The seepage facilities must be designed to store the entire runoff volume for a design storm compatible with the storm frequency used for design of the roadway drainage facilities or as directed by the Department. As a minimum, the seepage facilities shall be designed to store the increase in runoff volume from new impervious surfaces as long as adequate overflow conveyance paths are available to safely carry the larger flows to a stable discharge point.

Installation of seepage facilities can also satisfy runoff volume control and water quality concerns which may be required by an environmental permit.

Additional design guidelines are included in the NJDEP Stormwater Best Management Practices Manual.

2.3 Allowable Water Surface Elevation

Determine the allowable water surface elevation (AWS) at every site where a drainage facility will be constructed. The proposed drainage structure should cause a ponding level, hydraulic grade line elevation, or backwater elevation no greater than the AWS when the design flow is imposed on the facility. The AWS must comply with NJDEP requirements for locations that require a Stream Encroachment Permit. The AWS upstream of a proposed drainage facility at locations that do not require a Stream Encroachment Permit should not cause additional flooding outside the NJDOT property or acquired easements. An AWS that exceeds a reasonable limit may require concurrence of the affected property owner.

A floodplain study prepared by the New Jersey Department of Environmental Protection, the Federal Emergency Management Agency, the U.S. Army Corps of Engineers, or other recognized agencies will be available at some sites. The elevations provided in the approved study will be used in the hydraulic model.

The Table 2-1 presents additional guidelines for determining the AWS at locations where a Stream Encroachment Permit is not required.

Table 2-1
Allowable Water Surface (AWS)

Land Use or Facility	AWS
Residence	Floor elevation (slab floor), basement window, basement drain (if seepage potential is present)
Commercial Building (barn, store, warehouse, office building, etc.)	Same as for residence
Bridge	Low steel
Culvert	Top of culvert - New structure Outside edge of road - Existing structure
Levee	Min 1 foot below top of Levee
Dam	See NJDEP Dam Safety Standards
Channel	Min 1 foot below top of low bank
Road	Min 1 foot below top of grate or manhole rim for storm sewers

The peak 100-year water surface elevation for any new detention/retention facility must be contained within NJDOT property or acquired easements. No additional flooding shall result outside the NJDOT property or acquired easements.

2.4 Recurrence Interval

Select a flood recurrence interval consistent with Table 2-2:

Table 2-2
Recurrence Interval

Recurrence Interval	Facility Description
100-Year	Any drainage facility that requires a NJDEP permit for a non-delineated stream. For delineated watercourses contact the NJDEP Bureau of Floodplain Management.
50-Year	Any drainage structure that passes water under a freeway or interstate highway embankment, with a headwall or open end at each side of the roadway.
25-Year	Any drainage structure that passes water under a land service highway embankment, with a headwall or open end at each side of the roadway. Also, pipes along the mainline of a freeway or interstate highway that convey runoff from a roadway low point to the disposal point, a waterway, or a stormwater maintenance facility.
15-Year	Longitudinal systems and cross drain pipes of a freeway or interstate highway. Also pipes along mainline of a land service highway that convey runoff from a roadway low point to the disposal point, a waterway, or a stormwater maintenance facility.
10-Year	Longitudinal systems and cross drain pipes of a land service highway.

2.5 Increasing Fill Height Over Existing Structures

Investigate the structural adequacy of existing structures that will have additional loading as the result of a surcharge placement or construction loads.

2.6 Regulatory Compliance

Proposed construction must comply with the requirements of various regulatory agencies. Depending on the project location, these agencies could include, but are not limited to, the US Army Corps of Engineers, U. S. Coast Guard, the New Jersey Department of Environmental Protection, the Pinelands Commission, the Highlands Council and the Delaware and Raritan Canal Commission.

The NJDEP has adopted amendments to the New Jersey Pollution Discharge Elimination System (NJPDES) program to include a Construction Activity Stormwater General Permit (NJ 0088323). This program is administered by the NJ Department of Agriculture through the Soil Conservation Districts (SCD). Certification by the local SCD is not required for NJDOT projects. However, certification by the local SCD is required for non-NJDOT projects (i.e., a County is the applicant). A Request for Authorization (RFA) for a NJPDES Construction Stormwater Permit is needed only for non-NJDOT projects that disturb more than one (1) acre and must be submitted to the local SCD.

The NJDEP has adopted the New Jersey Stormwater Management Rule, N.J.A.C. 7.8. The Stormwater Management Rule governs all projects that provide for ultimately disturbing one (1) or more acres of land or increasing impervious surface by 0.25 acre or more. The following design and performance standards need to be addressed for any project governed by the Stormwater Management Rule:

- Nonstructural Stormwater Management Strategies, N.J.A.C. 7:8-5.3
  To the maximum extent possible, nonstructural stormwater BMPs shall be used to meet the requirements of the Stormwater Management Rule. If the design engineer determines that it is not feasible for engineering, environmental or safety reason to utilize nonstructural stormwater BMPs, structural BMPs may be utilized.
- Groundwater Recharge, N.J.A.C. 7:8-5.4(a)2
  For the project, the design engineer shall demonstrate either that the stormwater BMPs maintain 100% of the average annual preconstruction groundwater recharge volume for the site; or that the increase in stormwater runoff volume from pre-construction to post-construction for the 2-year storm is infiltrated. NJDEP has provided an Excel Spreadsheet to determine the project sites annual groundwater recharge amounts in both pre- and post-development site conditions. A full explanation of the spreadsheet and its use can be found in Chapter 6 of the New Jersey Stormwater Best Management Practices Manual. A copy of the spreadsheet can be downloaded from http://www.njstormwater.org.
- Stormwater Quantity, N.J.A.C. 7:8-5.4(a)3
  Stormwater BMPs shall be designed to do one of the following:
  1. The post-construction hydrograph for the 2-year, 10-year, and 100-year storm events do not exceed, at any point in time, the pre-construction runoff hydrographs for the same storm events.
  2. There shall be no increase, as compared to the pre-construction condition, in peak runoff rates of stormwater leaving the project

site for the 2-year, 10-year, and 100-year storm events and that the increased volume or change in timing of stormwater runoff will not increase flood damage at or downstream of the site. This analysis shall include the analysis of impacts of exiting land uses and projected land uses assuming full development under existing zoning and land use ordinances in the drainage area.

3. The post-construction peak runoff rates for the 2-year, 10-year, and 100-year storm events are 50%, 75%, and 80%, respectively, of the pre-construction rates. The percentages apply only to the post-construction stormwater runoff that is attributed to the portion of the site on which the proposed development or project is to be constructed.
4. Along tidal or tidally influenced waterbodies and/or in tidal floodplains, stormwater runoff quantity analysis shall only be applied if the increased volume of stormwater runoff could increase flood damages below the point of discharge. Tidal flooding is the result of higher than normal tides which in turn inundate low lying coastal areas. Tidal areas are not only activities in tidal waters, but also the area adjacent to the water, including fluvial rivers and streams, extending from the mean high water line to the first paved public road, railroad or surveyable property line. At a minimum, the zone extends at least 100 feet but no more than 500 feet inland from the tidal water body.

- Stormwater Quality, N.J.A.C. 7:8-5.5
  Stormwater BMPs shall be designed to reduce the post-construction load of TSS in stormwater runoff generated from the water quality storm by 80% of the anticipated load from the developed site. Section 12 and the Stormwater Best Management Practices Manual provide guidance in the planning and design of these facilities.
- Stormwater Maintenance Plan, N.J.A.C. 7:8-5.8
  The design engineer shall prepare a stormwater management facility maintenance plan in accordance with the New Jersey Stormwater Rule. At a minimum the maintenance plan shall include specific preventative maintenance tasks and schedules. Maintenance guidelines for stormwater management measures are available in the New Jersey Stormwater Best Management Practices Manual.

For projects located within the Pinelands or Highlands areas of the State, the design engineer should consult with the NJDEP to determine what additional stormwater management requirements may apply to the project. Additional information about the Pinelands can be found at http://www.state.nj.us/pinelands/, and information about the Highlands can be found at http://www.nj.gov/dep/highlands/.

On NJDOT projects, a RFA does not have to be sent to the SCD, but instead the environmental team sends a notification directly to the NJDEP. A RFA would have to be sent to the appropriate Soil Conservation District only for non-NJDOT projects (i.e. a County is the applicant).

The NJDOT Bureau of Environmental Services will provide guidance regarding project specific permit requirements. Guidance regarding NJDEP Stream Encroachment Permits is provided in Section 2.7.

2.7 Stream Encroachment

Stream Encroachment Permits for which the NJDOT is the applicant shall be processed in accordance with Section 13 of the NJDOT Procedures Manual and the following guidelines.

Applicability and specific requirements for all Stream Encroachment Permits may be found in the most recent Flood Hazard Area Control Act Rules as adopted by the New Jersey Department of Environmental Protection (NJDEP). Specific requirements for bridges and culverts are contained in N.J.A.C. 7.13 - 2.16.

In cases where the regulatory flood causes the water surface to overflow the roadway, the design engineer shall, by raising the profile of the roadway, by increasing the size of the opening or a combination of both, limit the water surface to an elevation equal to the elevation of the outside edge of shoulder. The design engineer is cautioned, however, to critically assess the potential hydrologic and hydraulic effects upstream and downstream of the project, which may result from impeding flow by raising the roadway profile, or from decreasing upstream storage and allowing additional flow downstream by increasing existing culvert openings. The design engineer shall determine what effect the resulting reduction of storage will have on peak flows and the downstream properties in accordance with the Flood Hazard Area Control Act Rules. Stormwater management facilities may be required to satisfy these requirements.

N.J.A.C. 7:13 - 2.3(b)1. indicates that the discharge for non-delineated watercourses is to be based on ultimate development in accordance with the current zoning plan. Hydraulic evaluation of existing roadway stream crossings may reveal that the water surface elevation for this discharge overtops the roadway. Compliance with both the bridge and culvert requirements presented in N.J.A.C. 7:13 - 2.16 and the NJDOT requirement to avoid roadway overtopping may require coordination between the agencies involved to achieve a reasonable design approach. In addition to the regulations listed above, the bridge and culvert design will be in compliance with the NJDEP's Technical Manual for Land Use Regulation Program, Bureaus of Inland and Coastal Regulations, Stream Encroachment Permits, which includes the following:

- Structures will pass the regulatory flood without increasing the upstream elevation of the flood profile by more than 0.2 feet if the structure is new or the upstream and downstream flood profile by more than 0.0 feet if the structure is a replacement for an existing structure.
- For new structures that result in lowering the downstream water surface elevation by 2 or 3 feet, the engineer must perform a routing analysis to verify that there are no adverse impacts further downstream.

Activities located along tidal waterbodies listed in the Flood Hazard Area Control Act Rules are not governed by NJDEP, Land Use Regulation Program, Stream Encroachment Section; however, a permit may be required from another unit of the NJDEP.

When a permit is required, the NJDOT Drainage Engineer shall be notified in writing. This notice shall include a USGS Location Map with the following information:

a. A title block identifying the project by name, the applicant, and the name of the quadrangle.
b. The limits of the project and point of encroachment shown in contrasting colors on the map.
c. The upstream drainage area contributing runoff shall be outlined for all streams and/or swales within or along the project.

   If the NJDOT Project Engineer, after consultation with NJDEP, determines that a pre-application meeting is desirable, the following engineering data may also be required for discussion at a NJDEP pre-application meeting.

d. A 1" = 30' scale plan with the encroachment location noted thereon.
e. In the case of a new or replacement structure or other type encroachment, the regulatory floodwater surface elevation as required for the review and analysis of the project impacts and permit requirements.

The design engineer is also required to determine whether a particular watercourse involved in the project is classified by the State as a Category One waterbody, and if so, shall design the project in accordance with the provisions at N.J.A.C. 7:9B-4. Projects involving a Category One waterbody shall be designed such that a 300-foot special water resource protection area is provided on each side of the waterbody. Encroachment within this 300-foot buffer is prohibited except in instances where preexisting disturbance exists. Where preexisting disturbance exists, encroachment is allowed, provided that the 95% TSS removal standard is met and the loss of function is addressed. More information on Category One Waters can be found in the NJDEP's web sites http://www.state.nj.us/dep or http://www.state.nj.us/dep/antisprawl/c1.html.

2.8 Soil Erosion and Sediment Control

The design for projects that disturb 5,000 or more square feet do not require plan certification from the local Soil Conservation District, but shall be prepared in accordance with the current version of the NJDOT Soil Erosion and Sediment Control Standards, including the required report. The Soil Erosion and Sediment Control Report shall include calculations and plans that address both temporary and permanent items for the engineering and vegetative standards. Calculations shall be shown for items that require specific sizing (e.g., rip rap, settling basins, etc.). Certification by the local Soil Conservation District is not required for NJDOT projects. Certification by the local Soil Conservation District is required for non-NJDOT projects (i.e., a County is the applicant).

3.0 Hydrology

3.1 Introduction

Hydrology is generally defined as a science dealing with the interrelationship between water on and under the earth and in the atmosphere. For the purpose of this section, hydrology will deal with estimating flood magnitudes as the result of precipitation. In the design of highway drainage structures, floods are usually considered in terms of peak runoff or discharge in cubic feet per second (cfs) and hydrographs as discharge per time. For drainage facilities which are designed to control volume of runoff, like detention facilities, or where flood routing through culverts is used, then the entire discharge hydrograph will be of interest. The analysis of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental to the design of drainage facilities. Errors in the estimates will result in a structure that is either undersized and causes more drainage problems or oversized and costs more than necessary.

In the hydrologic analysis for a drainage facility, it must be recognized that many variable factors affect floods. Some of the factors which need to be recognized and considered on an individual site by site basis include:

- rainfall amount and storm distribution,
- drainage area size, shape and orientation, ground cover, type of soil,
- slopes of terrain and stream(s),
- antecedent moisture condition,
- storage potential (overbank, ponds, wetlands, reservoirs, channel, etc.),
- watershed development potential, and
- type of precipitation (rain, snow, hail, or combinations thereof), elevation.

The type and source of information available for hydrologic analysis will vary from site to site. It is the responsibility of the design engineer to determine the information required for a particular analysis. This subsection contains hydrologic methods by which peak flows and hydrographs may be determined for the hydraulic evaluation of drainage systems of culverts, channels and median drains.

3.2 Selection of Hydrologic Methods

The following guidelines should be used to select the hydrology method for computing the design peak flow:

Table 3-1
Hydrologic Method

Size of Drainage Area	Hydrologic Method[‡]
Less than 20 Acres	Rational Formula or Modified Rational Method
Less than 5 Square Miles	NRCS[*] TR-55 Methodology
Greater than 1 Acre[ƒ]	NRCS[*] TR-20, HEC-1 Method, HEC-HMS or others[†]

‡ For all projects in certain areas south of the South Central flat inland and New Jersey Coastal Plain, the DELMARVA Unit Hydrograph shall be incorporated into the design procedure. Contact the local Soil Conservation District to determine if the DELMARVA unit hydrograph is to be used for the project.

* US Natural Resources Conservation Service (NRCS), formerly the US Soil Conservation Service (SCS).

ƒ These hydrologic models are not limited by the size of the drainage area. They are instead limited by uniform curve number, travel time, etc. Most of these limitations can be overcome by subdividing the drainage areas into smaller areas. See the appropriate users manual for a complete list of limitations for each hydrologic model.

† Many hydrologic models exist beyond those that are listed here. If a model is not included, then the design engineer should ensure that the model is appropriate and that approvals are obtained from the Department.

The peak flow from a drainage basin is a function of the basin’s physiographic properties such as size, shape, slope, soil type, land use, as well as climatological factors such as mean annual rainfall and selected rainfall intensities. The methods presented in the guideline should give acceptable predictions for the indicated ranges of drainage area sizes and basin characteristics.

Other hydrologic methods may be used only with the approval of the Department.

NOTE: If a watercourse has had a NJDEP adopted study prepared for the particular reach where the project is located, that study should be used for the runoff and water surface profiles. NJDEP does not accept FEMA studies, since the FEMA hydrologic models do not consider that the entire drainage area is to be fully developed. The design engineer should ensure that the hydrologic model used takes into account the NJDEP requirement that the entire upstream drainage area is to be considered fully developed.

Computation of peak discharge must consider the condition that yields the largest rate. Proper hydrograph combination is essential. It may be necessary to evaluate several different hydrograph combinations to determine the peak discharge for basins containing hydrographs with significantly different times for the peak discharge. For example, the peak discharge for a basin with a large undeveloped area contributing toward the roadway may result from either the runoff at the time when the total area reaches the roadway or the runoff from the roadway area at its peak time plus the runoff from the portion of the overland area contributing at the same time.

3.3 Rational Formula

The rational formula is an empirical formula relating runoff to rainfall intensity. It is expressed in the following form:

$$\mathbf{Q} = \mathbf{CIA}$$

Where:

Q = peak flow in cubic feet per second ft^3/s
C = runoff coefficient (weighted)
I = rainfall intensity in inches (in) per hour
A = drainage area in acres

A. **Basic Assumptions**

1. The peak rate of runoff (Q) at any point is a direct function of the average rainfall intensity (I) for the Time of Concentration (T_c) to that point.
2. The recurrence interval of the peak discharge is the same as the recurrence interval of the average rainfall intensity.
3. The Time of Concentration is the time required for the runoff to become established and flow from the most distant point of the drainage area to the point of discharge.

A reason to limit use of the rational method to small watersheds pertains to the assumption that rainfall is constant throughout the entire watershed. Severe storms, say of a 100-year return period, generally cover a very small area. Applying the high intensity corresponding to a 100-year storm to the entire watershed could produce greatly exaggerated flows, as only a fraction of the area may be experiencing such an intensity at any given time.

The variability of the runoff coefficient also favors the application of the rational method to small, developed watersheds. Although the coefficient is assumed to remain constant, it actually changes during a storm event. The greatest fluctuations take place on unpaved surfaces as in rural settings. In addition, runoff coefficient values are much more difficult to determine and may not be as accurate for surfaces that are not smooth, uniform and impervious.

To summarize, the rational method provides the most reliable results when applied to small, developed watersheds and particularly to roadway drainage design. The validity of each assumption should be verified for the site before proceeding.

B. **Procedure**

1. Obtain the following information for each site:
   a. Drainage area
   b. Land use (% of impermeable area such as pavement, sidewalks or roofs)
   c. Soil types (highly permeable or impermeable soils)
   d. Distance from the farthest point of the drainage area to the point of discharge
   e. Difference in elevation from the farthest point of the drainage area to the point of discharge
2. Determine the Time of Concentration (T_c). See Section 3.5. (Minimum T_c is 10 minutes).
3. Determine the rainfall intensity rate (I) for the selected recurrence intervals.
4. Select the appropriate C value.
5. Compute the design flow (Q = CIA).

The runoff coefficient (C) accounts for the effects of infiltration, detention storage, evapo-transpiration, surface retention, flow routing and interception. The product of C and the average rainfall intensity (I) is the rainfall excess of runoff per acre.

The runoff coefficient should be weighted to reflect the different conditions that exist within a watershed.

Example:

$$\mathbf{C_w = \frac{A_1C_1 + A_2C_2 \ldots A_NC_N}{A_1 + A_2 \ldots A_N}}$$

C. **Value for C**: Select the appropriate value for C from Table 3-2:

Table 3-2
Recommended Coefficient of Runoff Values
for Various Selected Land Uses

Land Use	Description	Hydrologic Soils Group			
		A	B	C	D
Cultivated Land	without conservation treatment	0.49	0.67	0.81	0.88
	with conservation treatment	0.27	0.43	0.67	0.67
Pasture or Range Land	poor condition	0.38	0.63	0.78	0.84
Meadow	good condition	---	0.25	0.51	0.65
	good condition	---	---	0.41	0.61
Wood or Forest Land	thin stand, poor cover, no mulch	---	0.34	0.59	0.70
	good cover	---	---	0.45	0.59
Open Spaces, Lawns, Parks, Golf Courses, Cemeteries					
Good Condition	grass cover on 75% or more	---	0.25	0.51	0.65
Fair Condition	grass cover on 50% to 75%	---	0.45	0.63	0.74
Commercial and Business Area	85% impervious	0.84	0.90	0.93	0.96
Industrial Districts	72% impervious	0.67	0.81	0.88	0.92
Residential	average % impervious				
Average Lot Size (acres)					
1/8	65	0.59	0.76	0.86	0.90
1/4	38	0.29	0.55	0.70	0.80
1/3	30	---	0.49	0.67	0.78
1/2	25	---	0.45	0.65	0.76
1	20	---	0.41	0.63	0.74
Paved Areas	parking lots, roofs, driveways, etc.	0.99	0.99	0.99	0.99
Streets and Roads	paved with curbs & storm sewers	0.99	0.99	0.99	0.99
	Gravel	0.57	0.76	0.84	0.88
	dirt	0.49	0.69	0.80	0.84

NOTE: Values are based on NRCS (formerly SCS) definitions and are average values.
Source: Technical Manual for Land Use Regulation Program, Bureau of Inland and Coastal Regulations, Stream Encroachment Permits, New Jersey Department of Environmental Protection

D. **Determination of Rainfall Intensity Rate (I)**: Determine the Time of Concentration (T_c) in minutes for the drainage basin. Refer to Section 3.5 for additional information.

Determine the value for rainfall intensity for the selected recurrence interval with a duration equal to the Time of Concentration from Figures 3-2 through 3-4. Rainfall Intensity "I" curves are presented in Figures 3-2 through 3-4. The curves provide for variation in rainfall intensity according to location, storm frequency, and Time of Concentration. Select the curve of a particular region where the site in question is located (see Figure 3-1 for determination of the particular region). For project that fall on the line or span more than one boundary, the higher intensity should be used for the entire project. The Regions can be defined by the following:

North Region: All Counties north of the Mercer and Monmouth County lines.

South Region: All Counties South of the Hunterdon, Somerset, and Middlesex County lines except for those areas located in the East Region.

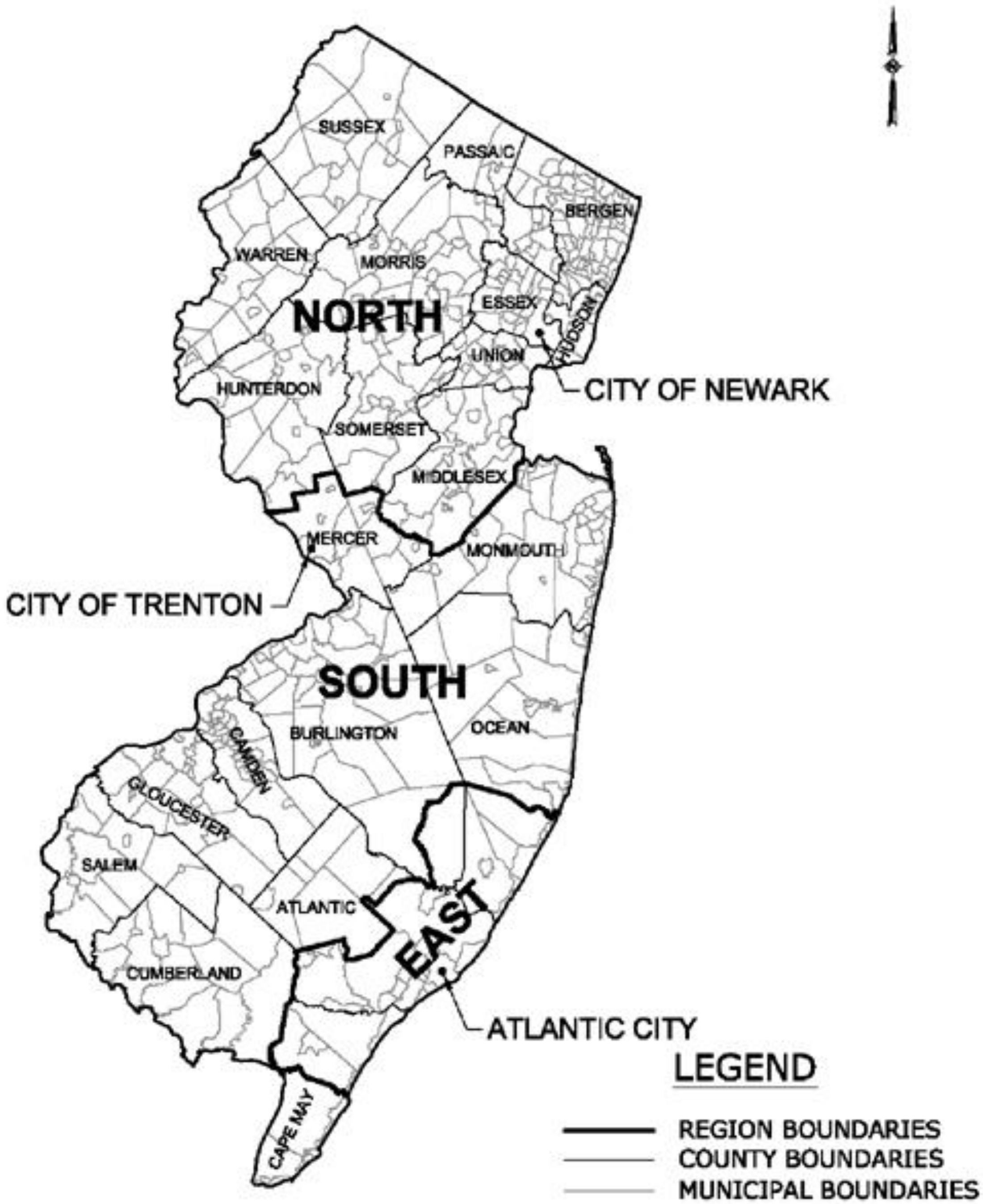
East Region: The eastern region is all municipalities east of the line delineated by the South municipal boundary of Sea Isle City, Cape May County to the South and Western boundary of Dennis Township, Cape May County to the western boundaries of Upper Township, Cape May County and Estell Manor City, Atlantic County to the West and North boundary of Weymouth Township, Atlantic County to the North boundary of Estell Manor City, Atlantic County to the North and East boundary of Weymouth Township, Atlantic County to the North boundary of Egg Harbor Township, Atlantic County to the East and North boundary of Galloway Township, Atlantic County to the North boundary of Port Republic City, Atlantic County to the East and North boundary of Bass River Township, Burlington County to the North boundary of Stafford Township, Ocean County to the East and North boundary of Harvey Cedars Boro, Ocean County.

The I-D-F curves provided were determined from data from the NOAA Atlas 14, Volume 2, Precipitation-Frequency of the United States. Development of Intensity-Duration-Frequency (I-D-F) curves is currently available in a number of computer programs. The programs develop an I-D-F curve based on user-supplied data or select the data from published data such as Hydro-35 or the aforementioned NOAA Atlas 14, Volume 2. Appendix A of HEC-12 contains an example of the development of rainfall intensity curves and equations.

Figure 3-1

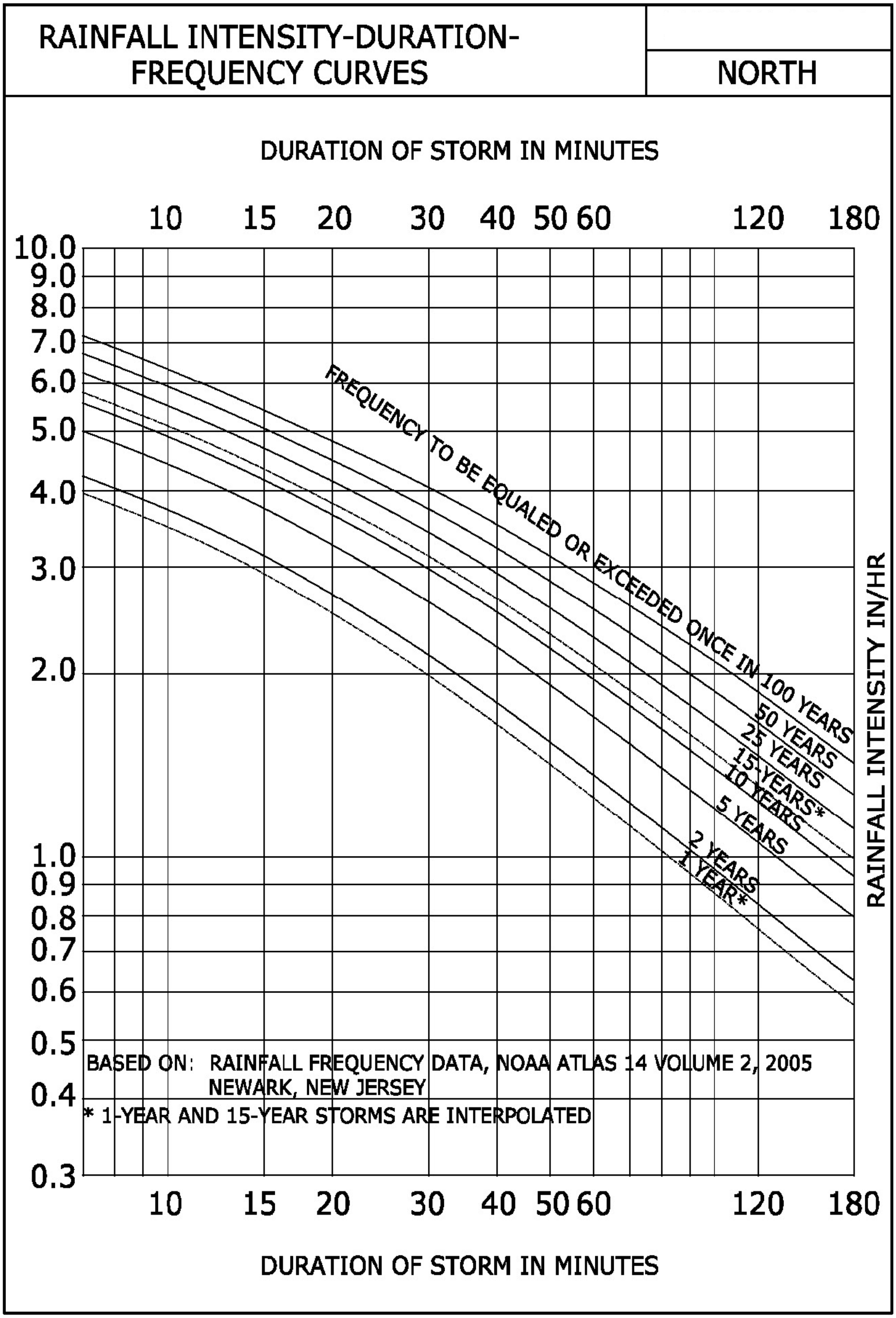
RAINFALL INTENSITY-DURATION-FREQUENCY CURVES

DELINEATED REGIONS WITH UNIFORM INTENSITY



DELINEATED REGIONS WITH UNIFORM INTENSITY

Figure 3-2



RAINFALL INTENSITY-DURATION-FREQUENCY CURVES
NORTH
DURATION OF STORM IN MINUTES
10 15 20 30 40 50 60 120 180
10.0 9.0 8.0 7.0 6.0 5.0 4.0 3.0 2.0 1.0 0.9 0.8 0.7 0.6 0.5 0.4 0.3
FREQUENCY TO BE EQUALED OR EXCEEDED ONCE IN 100 YEARS
50 YEARS
25 YEARS
15-YEARS*
10 YEARS
5 YEARS
2 YEARS
1 YEAR*
BASED ON: RAINFALL FREQUENCY DATA, NOAA ATLAS 14 VOLUME 2, 2005
NEWARK, NEW JERSEY
* 1-YEAR AND 15-YEAR STORMS ARE INTERPOLATED
RAINFALL INTENSITY IN/HR
DURATION OF STORM IN MINUTES

Figure 3-3

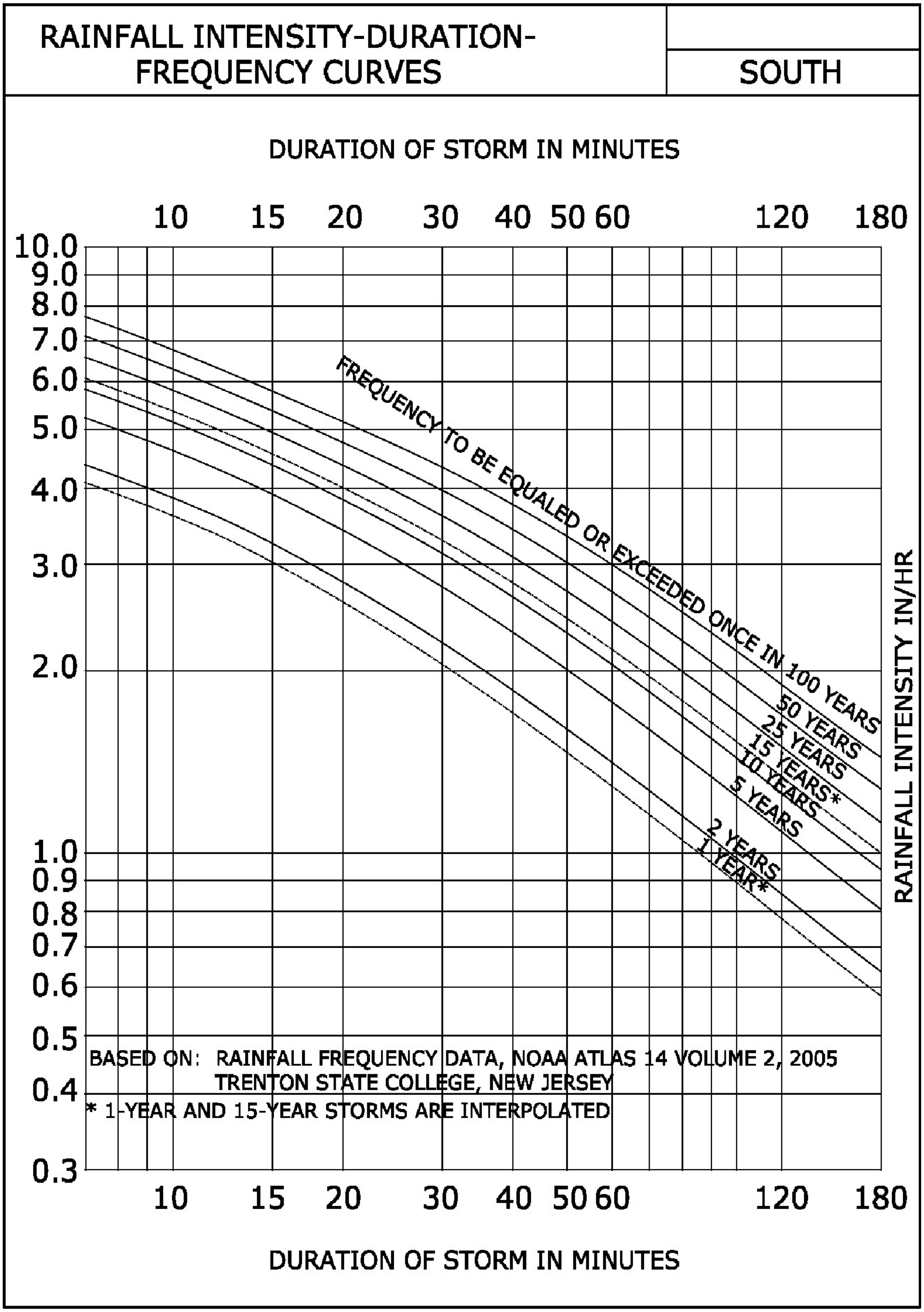
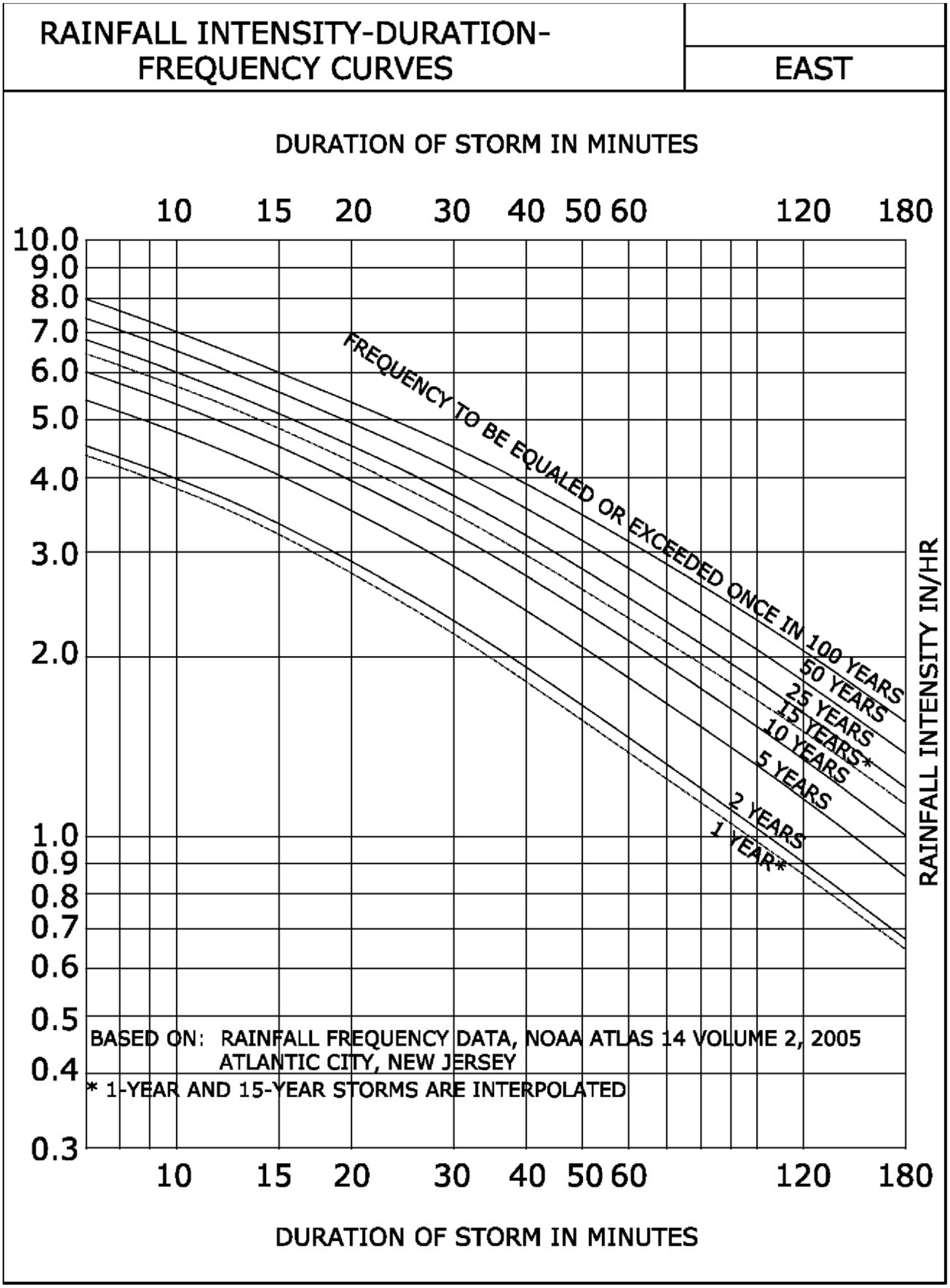

RAINFALL INTENSITY-DURATION-FREQUENCY CURVES
SOUTH
DURATION OF STORM IN MINUTES
10 15 20 30 40 50 60 120 180
10.0 9.0 8.0 7.0 6.0 5.0 4.0 3.0 2.0 1.0 0.9 0.8 0.7 0.6 0.5 0.4 0.3
FREQUENCY TO BE EQUALED OR EXCEEDED ONCE IN 100 YEARS
50 YEARS
25 YEARS
15 YEARS*
10 YEARS
5 YEARS
2 YEARS
1 YEAR*
RAINFALL INTENSITY IN/HR
BASED ON: RAINFALL FREQUENCY DATA, NOAA ATLAS 14 VOLUME 2, 2005
TRENTON STATE COLLEGE, NEW JERSEY
* 1-YEAR AND 15-YEAR STORMS ARE INTERPOLATED

Figure 3-4


RAINFALL INTENSITY-DURATION-
FREQUENCY CURVES
EAST
DURATION OF STORM IN MINUTES
10
15
20
30
40
50
60
120
180
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
FREQUENCY TO BE EQUALED OR EXCEEDED ONCE IN 100 YEARS
50 YEARS
25 YEARS
15 YEARS*
10 YEARS
5 YEARS
2 YEARS
1 YEAR*
BASED ON: RAINFALL FREQUENCY DATA, NOAA ATLAS 14 VOLUME 2, 2005
ATLANTIC CITY, NEW JERSEY
* 1-YEAR AND 15-YEAR STORMS ARE INTERPOLATED
RAINFALL INTENSITY IN/HR

Use of computer program-generated I-D-F curves shall be accepted provided the results match those obtained from Figures 3-2 through 3-4.

3.4 US Natural Resources Conservation Service (NRCS) Methodology

Techniques developed by the US Natural Resources Conservation Service (NRCS), formerly the US Soil Conservation Service (SCS) for calculating rates of runoff require the same basic data as the Rational Method: drainage area, a runoff factor, Time of Concentration, and rainfall. The NRCS approach, however, is more sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. With the NRCS method, the direct runoff can be calculated for any storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess. Details of the methodology can be found in the NRCS National Engineering Handbook, Section 4.

Two types of hydrographs are used in the NRCS procedure, unit hydrographs and dimensionless hydrographs. A unit hydrograph represents the time distribution of flow resulting from 1 inch of direct runoff occurring over the watershed in a specified time. A dimensionless hydrograph represents the composite of many unit hydrographs. The dimensionless unit hydrograph is plotted in nondimensional units of time versus time to peak and discharge at any time versus peak discharge.

Characteristics of the dimensionless hydrograph vary with the size, shape, and slope of the tributary drainage area. The most significant characteristics affecting the dimensionless hydrograph shape are the basin lag and the peak discharge for a specific rainfall. Basin lag is the time from the center of mass of rainfall excess to the hydrograph peak. Steep slopes, compact shape, and an efficient drainage network tend to make lag time short and peaks high; flat slopes, elongated shape, and an inefficient drainage network tend to make lag time long and peaks low.

The NRCS method is based on a 24-hour storm event which has a certain storm distribution. The Type III storm distribution should be used for the State of New Jersey. To use this distribution it is necessary for the user to obtain the 24-hour rainfall value for the frequency of the design storm desired. The 24-hour rainfall values for each county in New Jersey can be obtained from the NRCS and are contained in Table 3-3:

Table 3-3
New Jersey 24-Hour Rainfall Frequency Data
Rainfall amounts in Inches

County	Rainfall Frequency Data						
	1-Year	**2-Year**	**5-Year**	**10-Year**	**25-Year**	**50-Year**	**100-Year**
Atlantic	2.8	3.3	4.3	5.2	6.5	7.6	8.9
Bergen	2.8	3.3	4.3	5.1	6.3	7.3	8.4
Burlington	2.8	3.4	4.3	5.2	6.4	7.6	8.8
Camden	2.8	3.3	4.3	5.1	6.3	7.3	8.5
Cape May	2.8	3.3	4.2	5.1	6.4	7.5	8.8
Cumberland	2.8	3.3	4.2	5.1	6.4	7.5	8.8
Essex	2.8	3.4	4.4	5.2	6.4	7.5	8.7
Gloucester	2.8	3.3	4.2	5.0	6.2	7.3	8.5
Hudson	2.7	3.3	4.2	5.0	6.2	7.2	8.3
Hunterdon	2.9	3.4	4.3	5.0	6.1	7.0	8.0
Mercer	2.8	3.3	4.2	5.0	6.2	7.2	8.3
Middlesex	2.8	3.3	4.3	5.1	6.4	7.4	8.6
Monmouth	2.9	3.4	4.4	5.2	6.5	7.7	8.9
Morris	3.0	3.5	4.5	5.2	6.3	7.3	8.3
Ocean	3.0	3.4	4.5	5.4	6.7	7.9	9.2
Passaic	3.0	3.5	4.4	5.3	6.5	7.5	8.7
Salem	2.8	3.3	4.2	5.0	6.2	7.3	8.5
Somerset	2.8	3.3	4.3	5.0	6.2	7.2	8.2
Sussex	2.7	3.2	4.0	4.7	5.7	6.6	7.6
Union	2.8	3.4	4.4	5.2	6.4	7.5	8.7
Warren	2.8	3.3	4.2	4.9	5.9	6.8	7.8

Central to the NRCS methodology is the concept of the Curve Number (CN) which relates to the runoff depth and is itself characteristic of the soil type and the surface cover. CN's in Table 2-2 (*a* to *d*) of the TR-55 Manual (June 1986) represent average antecedent runoff condition for urban, cultivated agricultural, other agricultural, and arid and semiarid rangeland uses. Infiltration rates of soils vary widely and are affected by subsurface permeability as well as surface intake rates. Soils are classified into four Hydrologic Soil Groups (A, B, C, and D) according to their minimum infiltration rate. Appendix A of the TR-55 Manual defines the four groups and provides a list of most of the soils in the United States and their group classification. The soils in the area of interest may be identified from a soil survey report, which can be obtained from the local Soil Conservation District offices.

Several techniques have been developed and are currently available to engineers for the estimation of runoff volume and peak discharge using the NRCS methodology. Some of the more commonly used of these methods are summarized below:

A. **NRCS Technical Release 55** (TR-55): The procedures outlined in this document are the most widely used for the computation of tormwater runoff. This methodology is particularly useful for the comparison of pre- and post-development runoff rates and consequently for the design of control structures. There are basically two variations of this technique: the Tabular Hydrograph method and the Graphical Peak Discharge method.

1. **The Tabular Method** – This method provides an approximation of the more complicated NRCS TR-20 method. The procedure divides the watershed into subareas, completes an outflow hydrograph for each subarea and then combines and routes these hydrographs to the watershed outlet. This method is particularly useful for measuring the effects of changed land use in a part of the watershed. The Tabular method should not be used when large changes in the curve number occur among subareas or when runoff flow rates are less than 1345 ft^3/s for curve numbers less than 60. However, this method is sufficient to estimate the effects of urbanization on peak rates of discharge for most heterogeneous watersheds.

2. **Graphical Peak Discharge Method** – This method was developed from hydrograph analysis using TR-20, "Computer Program for Project Formulation-Hydrology" (NRCS 1983). This method calculates peak discharge using an assumed hydrograph and a thorough and rapid evaluation of the soils, slope and surface cover characteristics of the watershed. The Graphical method provides a determination of peak discharge only. If a hydrograph is required or subdivision is needed, the Tabular Hydrograph method should be used. This method should not be used if the weighted CN is less than 40.

For a more detailed account of these methods and their limitations the design engineer is referred to the NRCS TR-55 document.

B. **US Army Corps of Engineers HEC-1 Model**: This model is used to simulate watershed precipitation runoff processes during flood events. The model may be used to simulate runoff in a simple single basin watershed or in a highly complex basin with a virtually unlimited number of sub-basins and for routing interconnecting reaches. It can also be used to analyze the impact of changes in land use and detention basins on the downstream reaches. It can serve as a useful tool in comprehensive river basin planning and in the development of area-wide watershed management plans. The NRCS Dimensionless Unitgraph Option in the HEC-1 program shall be used. Other synthetic unit hydrograph methods available in HEC-1 can be used with the approval of the Department.

The HEC-1 model is currently supported by a number of software vendors which have enhanced versions of the original US Army Corps HEC-1 model. Refer to the available Program Documentation Manual for additional information.

C. **The NRCS TR-20 Model**: This computer program is a rainfall-runoff simulation model which uses a storm hydrograph, runoff curve number and channel features to determine runoff volumes as well as unit hydrographs to estimate peak rates of discharge. The dimensionless unit hydrographs from sub-basins within the watershed can be routed through stream reaches and impoundments. The TR-20 method may be used to analyze the impact of development and detention basins on downstream areas. The parameters needed in this method include total rainfall, rainfall distribution, curve numbers, Time of Concentration, travel time and drainage area.

3.5 Time of Concentration (T_c)

The Time of Concentration (T_c) is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. It may take a few computations at different locations within the drainage area to determine the most hydraulically distant point. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

T_c influences the shape and peak of the runoff hydrograph. Development usually decreases the T_c, thereby increasing the peak discharge, but T_c can be increased as a result of (a) ponding behind small or inadequate drainage systems, including storm drain inlets and road culverts, or (b) reduction of land slope through grading.

A. **Factors Affecting Time of Concentration and Travel Time**

1. **Surface Roughness**: One of the most significant effects of development on flow velocity is less retardance of flow. That is, undeveloped areas with very slow and shallow overland flow through vegetation become modified by development; the flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

2. **Channel Shape and Flow Patterns**: In small watersheds, much of the travel time results from overland flow in upstream areas. Typically, development reduces overland flow lengths by conveying storm runoff into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

3. **Slope**: Slopes may be increased or decreased by development, depending on the extent of site grading or the extent to which storm sewers and street ditches are used in the design of the tormwater management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers, street gutters, and diversions.

B. **Computation of Travel Time and Time of Concentration**: Water moves through a watershed as sheet flow, street/gutter flow, pipe flow, open channel flow, or some combination of these. Sheet flow is sometimes commonly referred to as overland flow. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection, review of topographic mapping and subsurface drainage plans.

A brief overview of methods to compute travel time for the components of the conveyance system is presented below.

1. **Rational Method**: Travel time for each flow regime shall be calculated as described below:

a. **Sheet Flow**: Using the slope and land cover type, determine the velocity from Figure 3-5. Sheet flow can only be computed for flow distances of 100 feet or less and for slopes of 24% or less
b. **Gutter Flow**: The gutter flow component of Time of Concentration can be computed using the velocity obtained from the Manning equation for the triangular gutter of a configuration and longitudinal slope as indicated by roadway geometry.
c. **Pipe Flow**: Travel time in a storm sewer can be computed using full flow velocities for the reach as appropriate.
d. **Open Channel Flow**: Travel time in an open channel such as a natural stream, swale, man-made ditch, etc., can be computed using the velocity obtained from the Manning equation or other acceptable computational procedure for open channel flow such as HEC-2.

Time of concentration (T_c) is the sum of travel time (T_t) values for the various consecutive flow segments:

$$\mathbf{T_c = T_{t1} + T_{t2} + \ldots T_{tm}}$$

where:

$\mathbf{T_c}$ = total Time of Concentration
$\mathbf{T_t}$ = travel time for each flow segment
m = number of flow segments

2. **TR-55**: The NRCS TR-55 method separates the flow into three basic segments: sheet flow, shallow concentrated flow, and open channel. The maximum length of sheet flow to be used is 150 feet. The open channel portion may be a natural channel, man-made ditch, or gutter flow along the roadway. The open channel portion time is determined by using the Manning's equation or other acceptable procedure for open channel flow such as HEC-2. Refer to TR-55, Chapter 3 for detailed information on the procedures.

The minimum Time of Concentration used shall be 10 minutes.

3.6 Flood Routing

The traditional design of storm drainage systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. This type of design may result in major drainage and flooding problems downstream. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream flows and often the cost of the downstream conveyance system. Flood routing shall be used to document the required storage volume to achieve the desired runoff control.

A hydrograph is required to accomplish the flood routing. A hydrograph represents a plot of the flow, with respect to time. The predicted peak flow occurs at the time, T_c. The area under the hydrograph represents the total volume of runoff from the storm. A hydrograph can be computed using either the Modified Rational Method (for drainage areas up to 20 acres) or the Soil Conservation Service 24-hour storm methodology described in previous sections. The Modified Rational Method is described in detail in Appendix A-5 of the NJDOT's Soil Erosion and Sediment Control Standards.

Storage may be concentrated in large basin-wide regional facilities or distributed throughout the watershed. Storage may be developed in roadway interchanges, parks and other recreation areas, small lakes, ponds and depressions. The utility of any storage facility depends on the amount of storage, its location within the system, and its operational characteristics. An analysis of such storage facilities should consist of comparing the design flow at a point or points downstream of the proposed storage site with and without storage. In addition to the design flow, other flows in excess of the design flow that might be expected to pass through the storage facility should be included in the analysis. The design criteria for storage facilities should include:

- release rate,
- storage and volume,
- grading and depth requirements,
- outlet works, and
- location.

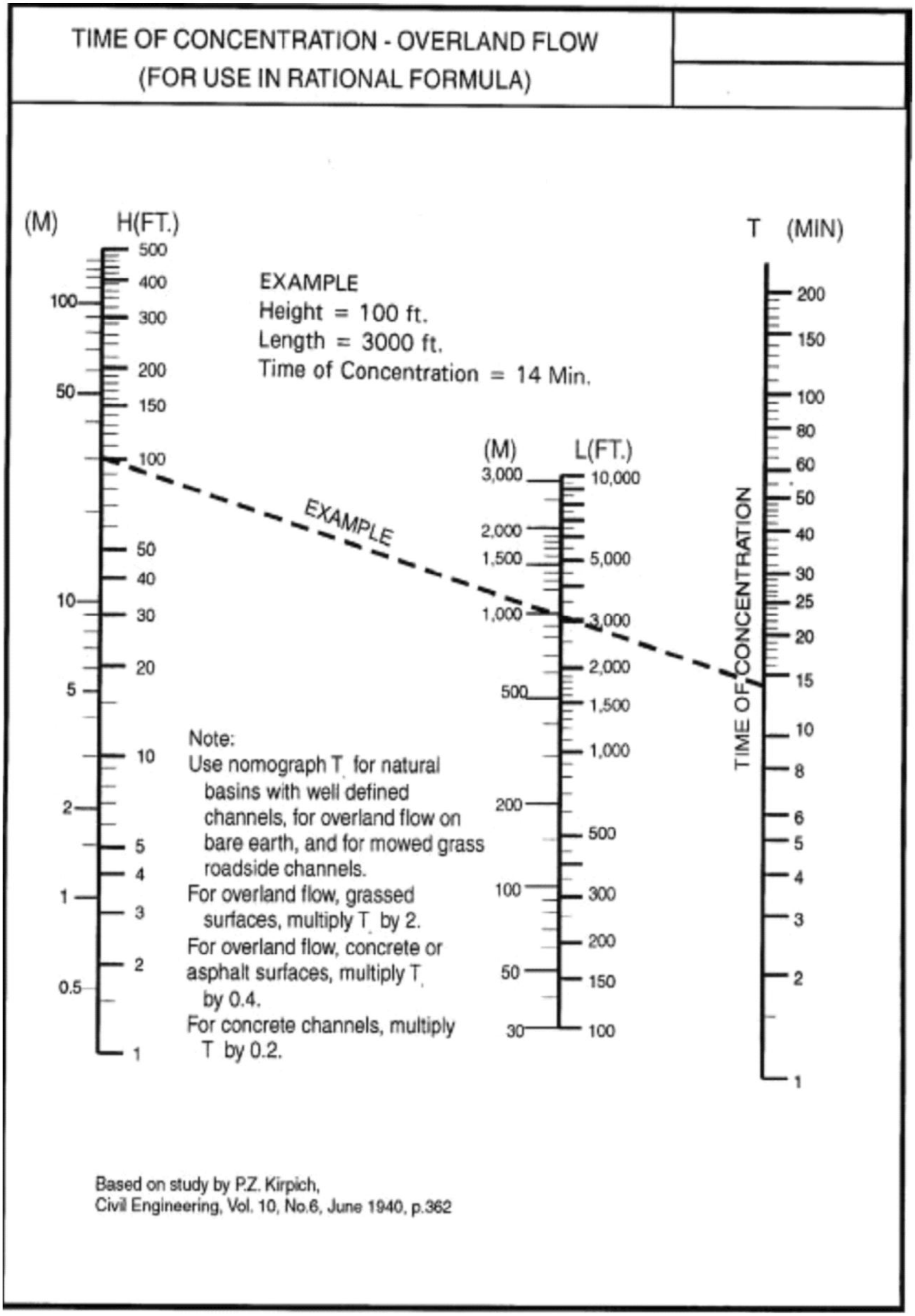
Control structure release rates shall be in accordance with criteria outlined in Section 2.0, Drainage Policy. Multi-stage control structures may be required to control runoff from different frequency events.

Storage volume shall be adequate to meet the criteria outlined in Section 2.2, Stormwater Management and Non-Point Source Pollution Control, to attenuate the post-development peak discharge rates or Section 2.3 to meet the allowable water surface elevation.

Outlet works selected for storage facilities typically include a principal spillway and an emergency overflow, and must be able to accomplish the design functions of the facility. Outlet works can take the form of combinations of drop inlets, pipes, weirs, and orifices. Standard acceptable equations such as the orifice equation ($Q = CA(2GH)^{1/2}$) or the weir equation ($Q = CL(H)^{3/2}$) shall be used to calculate stage-discharge relationships required for flood routings. The total stage-discharge curve shall take into account the discharge characteristics of all outlet works. Detailed information on outlet hydraulics can be found in the "Handbook of Hydraulics", by Brater and King.

Stormwater storage facilities are often referred to as either detention or retention facilities. For the purposes of this section, detention facilities are those that are designed to reduce the peak discharge and detain the quantity of runoff required to achieve this objective for a relatively short period of time. These facilities are designed to completely drain after the design storm has passed. Retention facilities are designed to contain a permanent pool of water. Since most of the design procedures are the same for detention and retention facilities, the term storage facilities will be used in this chapter to include detention and retention facilities.

Figure 3-5


TIME OF CONCENTRATION - OVERLAND FLOW
(FOR USE IN RATIONAL FORMULA)
(M) H(FT.)
EXAMPLE
Height = 100 ft.
Length = 3000 ft.
Time of Concentration = 14 Min.
EXAMPLE
(M) L(FT.)
T (MIN)
TIME OF CONCENTRATION
Note:
Use nomograph T for natural basins with well defined channels, for overland flow on bare earth, and for mowed grass roadside channels.
For overland flow, grassed surfaces, multiply T by 2.
For overland flow, concrete or asphalt surfaces, multiply T by 0.4.
For concrete channels, multiply T by 0.2.
Based on study by P.Z. Kirpich,
Civil Engineering, Vol. 10, No.6, June 1940, p.362

Routing calculations needed to design storage facilities, although not extremely complex, are time consuming and repetitive. Many reservoir routing computer programs, such as HEC-1, TR-20 and Pond-2, are available to expedite these calculations. Use of programs to perform routings is encouraged.

Section 11.0 and 12.0 contain standards related to stormwater management and quality control.

4.0 Channel Design

4.1 Introduction

Open channels, both natural and artificial, convey flood waters. Natural channels are crossed at highway sites and often need to be modified to accommodate the construction of a modern highway. Channels in the form of roadside ditches are added to the natural drainage pattern.

This part contains design methods and criteria to aid the design engineer in preparing designs incorporating these factors. Other open channel analysis methods and erosion protection information is also included.

4.2 Channel Type

The design of a channel is formulated by considering the relationship between the design discharge, the shape, slope and type of material present in the channel's bank and bed. Either grassed channels or non-erodible channels are typically used. The features of each are presented in the following narrative.

A. **Grassed Channels**: The grassed channel is protected from erosion by a turf cover. It is used in highway construction for roadside ditches, medians, and for channel changes of small watercourses. A grassed channel has the advantage of being compatible with the natural environment. This type of channel should be selected for use whenever possible.

B. **Non-erodible Channel**: A non-erodible channel has a lining that is highly resistant to erosion. This type of channel is expensive to construct, although it should have a very low maintenance cost if properly designed. Non-erodible lining should be used when stability cannot be achieved with a grass channel. Typical lining materials are discussed in the following narrative.

1. **Concrete Ditch Lining**: Concrete ditch lining is extremely resistant to erosion. Its principal disadvantages are high initial cost, susceptibility to failure if undermined by scour and the tendency for scour to occur downstream due to an acceleration of the flow velocity on a steep slope or in critical locations where erosion would cause extensive damage.

2. **Aggregate Ditch Lining**: This lining is very effective on mild slopes. It is constructed by dumping crushed aggregate into a prepared channel and grading to the desired shape. The advantages are low construction cost and self-healing characteristics. It has limited application on steep slopes where the flow will tend to displace the lining material.

3. **Alternative Linings**: Other types of channel lining such as gabion, or an articulated block system may be approved by the Department on a case-by-case basis, especially for steep sloped high velocity applications. HEC-11, Design of Riprap Revetment provides some design information on other types of lining.

4.3 Site Application

The design should consider site conditions as described below.

A. **Road Ditches**: Road ditches are channels adjacent to the roadway used to intercept runoff and groundwater occurring from areas within and adjacent to the right-of-way and to carry this flow to drainage structures or to natural waterways.

Road ditches should be grassed channels except where non-erodible lining is warranted. A minimum desirable slope of 0.5% should be used.

B. **Interceptor Ditch**: Interceptor ditches are located on the natural ground near the top edge of a cut slope or along the edge of the right-of-way to intercept runoff from a hillside before it reaches the backslope.

Interceptor ditches should be built back from the top of the cut slope, and generally at a minimum slope of 0.5% until the water can be emptied into a natural water course or brought into a road ditch or inlet by means of a headwall and pipe. In potential slide areas,   tormwater should be removed as rapidly as practicable and the ditch lined if the natural soil is permeable.

C. **Channel Changes**: Realignment or changes to natural channels should be held to a minimum. The following examples illustrate conditions that warrant channel changes:

1. The natural channel crosses the roadway at an extreme skew.
2. The embankment encroaches on the channel.
3. The natural channel has inadequate capacity.
4. The location of the natural channel endangers the highway embankment or adjacent property.

D. **Grade Control Structure**: A grade control structure allows a channel to be carried at a mild grade with a drop occurring through the structure (check dam).

4.4 Channel Design Procedure

The designed channel must have adequate capacity to convey the design discharge with 1 foot of freeboard.

Methods to design grass-lined and non-erodible channels are presented in the following narrative.

A. **Grassed Channel**: A grassed channel shall have a capacity designated in Section 2.4 – Recurrence Interval.

A non-erodible channel should be used in locations where the design flow would cause a grassed channel to erode.

The design of the grassed channel shall be in accordance with the NJDOT Soil Erosion and Sediment Control Standards Manual.

B. **Non-Erodible Channels**: Non-erodible channels shall have a capacity as designated in Section 2.4 – Recurrence Interval. The unlined portion of the channel banks should have a good stand of grass established so large flows may be sustained without significant damage.

The minimum design requirements of non-erodible channels shall be in accordance with the NJDOT Soil Erosion and Sediment Control Standards Manual where appropriate unless otherwise stated in this section.

1. **Capacity**: The required size of the channel can be determined by use of the Manning's equation for uniform flow. Manning's formula gives reliable results if the channel cross section, roughness, and slope are fairly constant over a sufficient distance to establish uniform flow. The Manning's equation is as follows:

$$\mathbf{Q} = \frac{\mathbf{1.486\ AR^{2/3}S^{1/2}}}{\mathbf{n}}$$

where

Q = Flow, cubic feet per second (ft^3/s)

n = Manning's roughness coefficient

Concrete, with surface as indicated:	Friction Factor Range
1. Formed, no finish	0.013-0.017
2. Trowel finish	0.012-0.014
3. Float finish	0.013-0.015
4. Float finish, some gravel on bottom	0.015-0.017
5. Gunite, good section	0.016-0.019
6. Gunite, wavy section	0.016-0.022

A = Area, square feet (ft^2)

P = Wetted perimeter, feet (ft)

R = Hydraulic radius (A/P)

S = Slope (ft/ft)

Design manuals such as Hydraulic Design Series No. 3 and No. 4 can be used as a reference for the design of the channels.

For non-uniform flow, a computer program, such as HEC-2, should be used to design the channel.

2. **Height of Lining**: The height of the lined channel should be equal to the normal depth of flow (D) based on the design flow rate, plus 1 foot for freeboard if possible.

3. **Horizontal Alignment**: Water tends to superelevate and cross waves are formed at a bend in a channel. If the flow is supercritical (as it will usually be for concrete-lined channels), this may cause the flow to erode the unlined portion of the channel on the outside edge of the bend. This problem may be alleviated either by superelevating the channel bed, adding freeboard to the outside edge, or by choosing a larger radius of curvature. The following equation relates freeboard to velocity, width, and radius of curvature:

$$H = \frac{V^2 W}{32.2 R_c}$$

where

H = Freeboard in feet (ft.)
V = Velocity in ft/s
W = Bottom width of channel in feet (ft.)
R_c = Radius of curvature in feet (ft.)

4. **Additional Design Requirements**:

   a. The minimum d_{50} stone size shall be 6 inches.
   b. The filter layer shall be filter fabric wherever possible.
   c. A 3 feet wide by 3 feet deep cutoff wall extending a minimum of 3 feet below the channel bed shall be provided at the upstream and downstream limits of the non-erodible channel lining.
   d. Additional design requirements may be required for permit conditions or as directed by the Department.
   e. Gradation of Aggregate Lining: The American Society of Civil Engineers Subcommittee recommends the following rules as to the gradation of the stone:

      (1) Stone equal to or larger than the theoretical d_{50}, with a few larger stones, up to about twice the weight of the theoretical size tolerated for reasons of economy in the utilization of the quarried rock, should make up 50 percent of the rock by weight.
      (2) If a stone filter blanket is provided, the gradation of the lower 50 percent should be selected to satisfy the filter requirements between the stone and the upper layer of the filter blanket.
      (3) The depth of the stone should accommodate the theoretically sized stone with a tolerance in surface in rule 1. (This requires tolerance of about 30 percent of the thickness of the stone.)
      (4) Within the preceding limitations, the gradation from largest to smallest sizes should be quarry run.

C. **Water Quality Channel Design**: The design of a water quality channel shall be in accordance with NJDOT and NJDEP requirements. Detailed requirements regarding water quality control is included in Section 12.0 Water Quality.

5.0 Drainage of Highway and Pavements

5.1 Introduction

Effective drainage of highway pavements is essential to maintenance of the service level of highways and to traffic safety. Water on the pavement slows traffic and contributes to accidents from hydroplaning and loss of visibility from splash and spray. Free-standing puddles which engage only one side of a vehicle are perhaps the most hazardous because of the dangerous torque levels exerted on the vehicle. Thus, the design of the surface drainage system is particularly important at locations where ponding can occur.

Runoff Collection and Conveyance System Type

Roadway runoff is collected in different ways based on the edge treatment, either curbed or uncurbed. Runoff collection and conveyance for a curbed roadway is typically provided by a system of inlets and pipe, respectively. Runoff from an uncurbed roadway, typically referred to as "an umbrella section", proceeds overland away from the roadway in fill sections or to roadside swales or ditches in roadway cut sections.

Conveyance of surface runoff over grassed overland areas or swales and ditches allows an opportunity for the removal of contaminants. The ability of the grass to prevent erosion is a major consideration in the design of grass-covered facilities. Use of an "umbrella" roadway section may require additional ROW.

Areas with substantial development adjacent to the roadway, particularly in urbanized areas, typically are not appropriate for use of a roadway "umbrella" section.

The decision to use an "umbrella" section requires careful consideration of the potential problems. Benefits associated with "umbrella" sections include cost savings and eliminating the possibility of vehicle vaulting. "Umbrella" sections used on roadways with higher longitudinal slopes have been found to be prone to berm washouts. Debris build-up along the edge of the roadway creates a curb effect that prevents sheet flow and directs the water along the edge of the roadway. This flow usually continues along the edge until a breach is created, often resulting in substantial erosion. Some situations may also warrant installing inlets along the edge of an "umbrella" section to pick up water which may become trapped by berm buildup or when snow is plowed to the side of the roadway and creates a barrier that will prevent sheet flow from occurring.

Bermed sections are designed with a small earth berm at the edge of the shoulder to form a gutter for the conveyance of runoff. Care should be taken to avoid earth berms on steep slopes that would cause erosive velocities yielding berm erosion.

An "umbrella" section should be used where practical. However, low points at umbrella sections should have inlets and discharge pipes to convey the runoff safely to the toe of slope. A Type "E" inlet and minimum 15 inch diameter pipe shall be used to drain the low point. Snow inlets (see Section 5.12) shall be provided where the pile up of snow in the berm area prevents drainage of the low points.

"Umbrella" sections should be avoided on land service roadways where there are abutting properties and driveways.

Slope treatment shall be provided at all low points of umbrella sections and all freeway and interstate projects to provide erosion protection (see NJDOT Standard Details).

5.3 Types of Inlets Used by NJDOT

Inlet grate types used by NJDOT consist of two types, combination inlets (with a curb opening), and grate inlets (without a curb opening) as shown on the current standard details as summarized below:

1. Combination Inlets B, B1, B2, C, D1, D2
2. Grate Inlets A, B Mod., B1 Mod., B2 Mod., E, E1, E2, ES

Inlets Type B1, B2, B1 Modified, B2 Modified, E1 or E2 will be used as necessary to accommodate large longitudinal pipes. A special inlet shall be designed, with the appropriate detail provided in the construction plans, and the item shall be designated "Special Inlet", when the pipe size requires a structure larger than a Type B2, B2 Modified or E2. A special inlet shall also be designed, with the appropriate detail provided in the construction plans, and the item shall be designated "Special Inlet", when the transverse pipe size requires a structure larger than the standard inlet types.

Drainage structure layout should minimize irregularities in the pavement surface. Manholes should be avoided where practicable in the traveled way and shoulder. An example is a widening project where inlets containing a single pipe should be demolished and the pipe extended to the proposed inlet, as opposed to placing a slab with a standard manhole cover or square frame with round cover on the existing inlet and extending the pipe to the new inlet.

5.4 Flow in Gutters (Spread)

The hydraulic capacity of a gutter depends on its cross-section geometry, longitudinal grade, and roughness. The typical curbed gutter section is a right triangular shape with the curb forming the vertical leg of the triangle. Design shall be based on the following frequencies:

Recurrence Interval	Facility Description
15-Year	Freeway or interstate highway
10-Year	Land service highway

The Manning equation has been modified to allow its use in the calculation of curbed gutter capacity for a triangular shaped gutter. The resulting equation is:

$$Q = (0.56/n)(S_x^{5/3})(S_o^{1/2})\,T^{8/3} \quad (1)$$

where

Q = rate of discharge in ft^3/s

n = Manning's coefficient of gutter roughness (Table 5-1)

S_x = cross slope, in ft/ft

S_o = longitudinal slope, in ft/ft

T = spread or width of flow in feet

The relationship between depth of flow **(y)**, spread **(T)**, and cross slope **(S_x)** is as follows:

$$y = TS_x$$, depth in gutter, at deepest point in feet.

Table 5-1
Roughness Coefficients
Manning's "n"

Street and Expressway Gutters		
a.	Concrete gutter troweled finish	0.012
b.	Asphalt pavement 1) Smooth texture 2) Rough texture	 0.013 0.016
c.	Concrete gutter with asphalt pavement 1) Smooth 2) Rough	 0.013 0.015
d.	Concrete pavement 1) Float finish 2) Broom finish	 0.014 0.016
e.	Brick	0.016
For gutters with small slope where sediment may accumulate, increase all above values of "n" by 0.002.		

5.5 Limits of Spread

The objective in the design of a drainage system for a highway pavement section is to collect runoff in the gutter and convey it to pavement inlets in a manner that provides reasonable safety for traffic and pedestrians at a reasonable cost. As spread from the curb increases, the risks of traffic accidents and delays and the nuisance and possible hazard to pedestrian traffic increase. The following shall be used to determine the allowable spread.

1. Width of inside and outside shoulder along interstate and freeway mainline
2. 1/3 width of ramp proper, 1/3 of live lanes next to curb and lanes adjacent to inside and outside shoulders on land service roads
3. 1/2 width of acceleration or deceleration lanes

The limits of spread are summarized in Table 5-2.

Table 5-2
Limits of Spread

Lane Configuration	Interstate and Freeways	Land Service Roads
Live Lanes next to Shoulder (inside & outside)	Full Shoulder	1/3 Width of Lane
Live Lanes next to Curb	---	1/3 Width of Lane
Ramp Proper	1/3 Width of Ramp	1/3 Width of Ramp
Accel/Decel Lanes	1/2 Width of Lane	1/2 Width of Lane

5.6 Inlets

There are separate design standards for grates in pavement or other ground surfaces, and for curb opening inlets. Each standard is described below. These standards help prevent certain solids and floatables (e.g., cans, plastic bottles, wrappers, and other litter) from reaching the surface waters of the State. For new roadway projects and reconstruction of existing highway, storm drain inlets must be selected to meet the following design requirements:

A. Grates in Pavement or Other Ground Surfaces

Many grate designs meet the standard. The first option (especially for storm drain inlets along roads) is simply to use the Department's bicycle safe grate. The other option is to use a different grate, as long as each "clear space" in the grate (each individual opening) is:

- No larger than seven (7.0) square inches; or
- No larger than 0.5 inches (½ inch) across the smallest dimension (length or width).

B. Curb-Opening Inlets

If the storm drain inlet has a curb opening, the clear space in that curb opening (or each individual clear space, if the curb opening has two or more clear spaces) must be:

- No larger than two (2.0) inches across the smallest dimension (length or width) - many curb opening inlets installed in recent years meet this criterion; or
- No larger than seven (7.0) square inches

C. Exemptions

The requirements for Grates in Pavement or Other Ground Surfaces or Curb-Opening Inlets do not apply in certain circumstances. See the New Jersey Department of Environmental Protection Highway Agency Stormwater Guidance and "Stormwater Management Rule", N.J.A.C. 7.8 for a complete list of exemptions.

Storm Drain inlets that are located at rest areas, service areas, maintenance facilities, and along streets with sidewalks operated by the Department are required to have a label placed on or adjacent to the inlet. The label must contain a cautionary message about dumping pollutants. The message may be a short phrase and/or graphic approved by the Department. The message may be a short phrase such as “The Drain is Just for Rain”, “Drains to [Local Waterbody]”, “No Dumping. Drains to River”, “You Dump it, You Drink it. No Waste Here”. or it may be a graphic such as a fish. Although a stand-alone graphic is permissible, the Department strongly recommends that a short phrase accompany the graphic.

The hydraulic capacity of an inlet depends on its geometry and gutter flow characteristics. Inlets on grade demonstrate different hydraulic operation than inlets in a sump. The design procedures for inlets on grade are presented in Section 5.7, "Capacity of Gutter Inlets on Grade". The design procedures for inlets in a sump are presented in Section 5.8, "Capacity of Grate Inlets at Low Points". Proper hydraulic design in accordance with the design criteria maximizes inlet capture efficiency and spacing. The inlet efficiency should be a minimum of 75%.

5.7 Capacity of Gutter Inlets on Grade

Collection capacity for gutter inlets on grade shall be determined using the following empirical equation:

$$\mathbf{Q_i = 16.88y^{1.54}(S^{0.233}/S_x^{0.276})}$$

where

Q_i = flow rate intercepted by the grate (ft^3/s)

y = gutter depth (ft) for the approach flow

S = longitudinal pavement slope

S_x = transverse pavement slope

The equation was developed for the standard NJDOT Type “A” grate configuration and is to be used for all inlet grate types without modification.

An alternative procedure, that yields results reasonably close to those obtained by using the runoff collection capacity equation presented above, is to compute the collection capacity in accordance with the procedures presented in Federal Highway Administration, Hydraulic Engineering Circular No. 12 (HEC-12) “Drainage of Highway Pavements” using the following parameter values:

Grate type P-1-7/8-4

Constant representative splash-over velocity of 5.77 ft/s

Constant effective grate length of 2.66 feet

All other parameter values for use in this procedure are as stated in HEC-12. Use of computer programs is encouraged to perform the tedious hydraulic capacity calculations. HEC-12 contains useful charts and tables. The HEC-12 procedure is also incorporated in a number of computer software programs.

5.8 Capacity of Grate Inlets at Low Points

Hydraulic evaluation of the bicycle safe grate reveals that the grate functions as a weir for approach flow depths equal to or less than 9 inches and as an orifice for greater depths. Procedures to compute the collection capacity for each condition are presented separately below.

Weir Flow

Collection capacity shall be determined using equation 17 presented on page 69 of HEC-12:

$$\mathbf{Q_i = C_w P y^{1.5}}$$

where

$\mathbf{Q_i}$ = flow rate intercepted by the grate (ft^3/s)

$\mathbf{C_w}$ = weir coefficient

$\mathbf{P}$ = perimeter around the open area of the grate (as shown on chart 11, on page 71 of HEC-12)

$\mathbf{y}$ = depth (ft) for the approach flow

The weir flow coefficient is 3.0. The perimeter around the open area for various NJDOT bicycle safe grate configurations and the resultant product of C_wP are summarized as follows:

Inlet Type	Perimeter* (ft)	C_wP*
A, B Mod., B1 Mod., B2 Mod.	5.28	15.84
B, B1, B2, C, D1, D2, E	6.96	20.88
ES	5.18	15.54

*Type "B", "C", and "D" inlets have a curb opening that allows runoff to enter the inlet even when debris partly clogs the grate. The equations must be modified for use with inlets that do not have a curb opening to account for reduced interception capacity resulting from debris collecting on the grate. The perimeter around the open area of the grate (P) used in the weir equation should be divided in half for inlets without a curb opening. The perimeter and resultant product of C_wP for inlet types "A", "B Mod.", "E" and "ES" shown in the table reflect this modification.

Orifice Flow

Collection capacity shall be determined using equation 18 presented on page 69 of HEC-12 (1984):

$$Q_i = C_o A_o (2gy)^{0.5}$$

where

Q_i = flow rate intercepted by the grate (ft^3/s)

C_o = orifice coefficient

A_o = clear opening area of a single grate

y = depth (ft) for the approach flow

g = gravitational acceleration of 32.2 ft/sec^2

The orifice flow coefficient is 0.67. The clear opening area and resultant product of C_oA_o for various NJDOT bicycle safe grate configurations are summarized as follows:

Inlet Type	Clear Opening Area* (ft^2)	C_oA_o*
A, B Mod., B1 Mod., B2 Mod.	1.45	0.97
B, B1, B2, C, D1, D2, E, ES	2.90	1.94

*Type "B" "C", and "D" inlets have a curb opening that allows runoff to enter the inlet even when debris partly clogs the grate. The equations must be modified for use with inlets that do not have a curb opening to account for reduced interception capacity resulting from debris collecting on the grate. The clear opening area of the grate (A_o) used in the orifice equation should be divided in half for inlets without a curb opening. The clear opening area and resultant product of C_oA_o for inlet types" A", "B Mod.", "E" and "ES" reflect this modification.

5.9 Location of Inlets

Proper inlet spacing enhances safety by limiting the spread of water onto the pavement. Proper hydraulic design in accordance with the design criteria maximizes inlet capture efficiency and spacing. Inlets should be located primarily as required by spread computations. See Section 5.7 and Section 5.8. Additional items to be considered when locating inlets include:

A. Low points in gutter grade. Adjust grades to the maximum extent possible to ensure that low point do not occur at driveways, handicap accessible areas, critical access points, etc.
B. At intersections and ramp entrances and exits to limit the flow of water across roadways.
C. Upgrade of cross slope rollover at the point fifty (50) feet upstream of the 0% cross slope.
D. Upgrade of all bridges and downgrade of bridges in fill section before the end of curb where the curb is not continuous.
E. Along mainline and ramps as necessary to limit spread of runoff onto roadway in accordance with Section 5.5.

5.10 Spacing of Inlets

The spacing of inlets along the mainline and ramps is dependent upon the allowable spread and the capacity of the inlet type selected. Maximum distance between inlets is 400 feet. The procedure for spacing of inlets is as follows:

A. Calculate flow and spread in the gutter. Tributary area is from high point to location of first inlet. This location is selected by the design engineer. Overland areas that flow toward the roadway are included.
B. Place the first inlet at the location where spread approaches the limit listed in Section 5.9.
C. Calculate the amount of water intercepted by the inlet, check the grate efficiency. This efficiency should be a minimum of 75%.
D. The water that bypasses the first inlet should be included in the flow and spread calculation for the next inlet.
E. This procedure is repeated to the end of the system. Sample calculations are presented in Section 13.0.

5.11 Depressed Gutter Inlet

Placing the inlet grate below the normal level of the gutter increases the cross-flow towards the opening, thereby increasing the inlet capacity. Also, the downstream transition out of the depression causes backwater which further increases the amount of water captured.

A. **Locations of Depressed Inlets**

1. All inlets in shoulders greater than 4 feet wide.
2. All inlets in one-lane, low speed ramps.
3. Inlets will not be depressed next to a riding lane, acceleration lane, deceleration lane, two-lane ramps, and direct connection ramps or within the confines of a bridge approach and transition slab.

B. **Limits of Depression**

1. Begin depression a distance of 4 feet upgrade of inlet.
2. End depression a distance of 2 feet downgrade of inlet.
3. Begin depression 4 feet out from gutter line.
4. Depth of depression, 2 inches below projected gutter grade.

See NJDOT Standard Roadway Construction / Traffic Control / Bridge Construction Details; CD-603-3, Method of Depressing Inlets at Shoulders.

C. **Spacing of Depressed Inlets**

Use the same procedure as described in Section 5.9. This method will give a conservative distance between inlets; however, this will provide an added safety factor and reduce the number of times that water will flow on the highway riding lanes when the design storm is exceeded.

5.12 Snow Melt Control

Roadway safety can be enhanced by snow melt runoff control. Collection of snow melt runoff is important on the high side of superelevated roadways and at low points. A discussion of each situation and the design approach is outlined below.

A. **Snowmelt Collection on High Side of Superelevation**

Collection of snow melt on the high side of a superelevated section from roadway and berm areas before it crosses the roadway prevents icing during the freeze-thaw process. Therefore, a safety offset or small shoulder (4 feet wide) sloped back towards the curb at a rate of 6% will provide a means to convey the snow melt water to inlets installed for this purpose. The snow melt inlets should be placed along the outer curbline at the upstream side of all intersections and at convenient cross drain locations. The snow melt inlets should be connected to the drainage system with a 15 inch diameter pipe to the trunk storm sewer. The small shoulder and snow inlets will not be designed to control stormwater runoff but shall be designed to handle only the small amount of expected flow from the snowmelt.

B. **Snowmelt Collection at Low Points**

Collection of snowmelt is important at low points where the pile-up of snow over existing inlets prevents draining of snowmelt and runoff off the edge of road. The addition of inlets placed away from the edge of curb and beyond anticipated snow piles provides a means to drain snowmelt.

Snow inlets are required at all roadway profile low points. All snow inlets shall be Type "E". Snow inlets shall not be depressed.

Snow inlets shall be provided in the shoulder immediately adjacent to the travel lane without encroaching on the travel lane.

Snow inlets shall not be installed in shoulders where the width is so narrow that placement of a snow inlet will encroach upon the inlet at the curb.

Pipes draining snow inlets shall be a minimum 15 inches diameter, sloped at a 1% minimum grade wherever possible.

5.13 Alternative Runoff Collection Systems

Standard roadway inlets are used to collect runoff on curbed roadways. Compliance with the established spread criteria for roadways with flat grades typically requires many inlets, usually installed at close intervals. Use of alternative collection systems such as trench drains may be appropriate to reduce the number of inlets required to satisfy the spread criteria. Therefore, use of trench drains for runoff collection on roads with flat grades may be warranted. The trench drain should be located upstream of the inlet to which it connects. The length of trench drain should provide the capture capacity that together with the inlet limits bypass at the inlet to zero.

Trench drain capture computations require consideration of both frontal and side flow capture. Frontal flow captured by the narrow trench drain is small and is, therefore, disregarded. Side flow into the trench drain is similar to flow into a curb opening inlet. Hydraulic evaluation procedures for curb opening inlets are described in FHWA HEC-12. Side flow is computed using the procedures for curb opening inlets presented in FHWA HEC-12. The trench drain must be long enough to intercept the bypass after frontal flow plus the additional runoff contributed by the roadway for the length of the trench drain. The process includes the following steps:

A. Compute the total runoff to the inlet.
B. Compute the frontal flow captured by inlet with no bypass allowed for the spread limited to the width of the grate. The runoff to be intercepted by the trench drain is the total runoff minus the runoff captured by the inlet.
C. Compute the length of trench drain required to capture the discharge using the curb opening inlet procedures in FHWA HEC-12. The computed length shall be multiplied by two to reflect inefficiencies due to clogging.

Maintenance requirements for trench drains should also be considered in the evaluation of trench drains. Use of a trench drain system should be discussed with the Department early in the design process with recommendations submitted prior to completion of the Initial Submission.

6.0 Storm Drains

6.1 Introduction

A storm drain is that portion of the roadway drainage system that receives runoff from inlets and conveys the runoff to some point where it can be discharged into a ditch, channel, stream, pond, lake, or pipe. This section contains the criteria and procedures for the design of roadway drainage systems.

6.2 Criteria for Storm Drains

Storm drains shall be designed using the following criteria where applicable:

A. Minimum pipe size is 15 inches.
B. Minimum pipe size is 18 inches downstream of mainline lowpoints.
C. Storm sewer pipe materials for proposed systems typically include concrete, corrugated metal, aluminum alloy and Smooth interior High Density Polyethylene (HDPE). Manning's roughness coefficient "n" for concrete and HDPE pipe is .012. Manning's roughness coefficient values for corrugated metal and aluminum alloy pipe are presented in Table 6-1. Manning's roughness coefficients for other materials occasionally encountered are indicated below:

Table 6-1.
Manning's roughness coefficients - Other

Manning's Roughness Coefficient, "n"		
Closed Culverts:		
Vitrified clay pipe		0.012-0.014
Cast-iron pipe, uncoated		0.013
Steel pipe		0.009-0.011
Brick		0.014-0.017
Monolithic concrete:		
1.	Wood forms, rough	0.015-0.017
2.	Wood forms, smooth	0.012-0.014
3.	Steel forms	0.012-0.013
Cemented rubble masonry walls:		
1.	Concrete floor and top	0.017-0.022
2.	Natural floor	0.019-0.025
Laminated treated wood		0.015-0.017
Vitrified clay liner plates		0.015

D. Design to flow full, based on uniform flow.
E. Minimum self-cleaning velocity of 2.5 ft/sec. should be maintained wherever possible.

Table 6-2
Values of Coefficient of Manning's Roughness (n) for Corrugated Metal and Aluminum Alloy Pipe (Unpaved Inverts and Unlined Pipe)

Annular 2 2/3" x 1/2" Corrugations	**Helical Corrugations***							
All Diameters	1 1/2" x 1/4"		2 2/3" x 1/2"					
	8 inch	10 inch	12 inch	18 inch	24 inch	36 inch	48 inch	60 inch & Larger
0.024	0.012	0.014	0.011	0.013	0.015	0.018	0.020	0.021
Annular 3" x 1"	Helical - 3" x 1"							
	48 inch	54 inch	60 inch	66 inch	72 inch	78 inch & Larger		
0.027	0.023	0.023	0.024	0.025	0.026	0.027		
Annular 5" x 1"	Helical - 5" x 1"							
	54 inch	60 inch	66 inch	72 inch	78 inch & Larger			
0.025	0.022	0.023	0.024	0.025	0.027			

*The "n" values shown above for helical corrugations apply only when spiral flow can be developed. The design engineer must assure himself/herself that spiral flow will occur in his/her design situation. Spiral flow will not occur when the following conditions exist, in which case the "n" value for annular corrugations is to be used:

1. Partly full flow
2. Non-circular pipes, such as pipe arches
3. When helical C.M.P. is lined or partly lined
4. Short runs less than 20 diameters long

Pipe arches have the same roughness characteristics as their equivalent round pipes

F. Structural design (class or gauge) of storm drains shall be in accordance with current AASHTO Standard Specifications for Highway Bridges. Structural evaluation of storm drains may be made using the following texts/references where appropriate if they are consistent with AASHTO:

   1. Concrete: Concrete Pipe Design Manual American Concrete Pipe Association
   2. Corrugated Metal Pipe: Handbook of Steel Drainage and Highway Construction Products
   3. Aluminum Alloy Pipes (as recommended by manufacturer)
   4. Smooth interior HDPE (as recommended by manufacturer)

G. Maximum grade on which concrete pipe should be placed is 10%.
H. HDPE, shall be used for pipe lengths outside of the roadbed only. HDPE pipe is not allowed for lateral pipes or for the outlet pipe to a receiving watercourse or water body. The density of polyethylene pipe is less than

water, therefore when wet conditions are expected, polyethylene pipe will float and should not be specified. End sections for HDPE shall be concrete.

I. Flared end-sections should be used whenever and wherever possible, for concrete and metal pipe.
J. Pipe sizes should not decrease in the downstream direction even though an increase in slope would allow a smaller size.
K. Pipe slopes should conform to the original ground slope so far as possible to minimize excavation.
L. For durability, the minimum thickness for steel pipe is 14 gauge and for aluminum alloy pipe is 16 gauge. In extremely corrosive areas and where high abrasion can be expected the design engineer shall determine whether heavier gauges should be used.
M. Material types: See Figure 6-1.

   a. Concrete, HDPE, and Aluminum Alloy pipe may be used in the shaded area.
   b. Concrete, HDPE, Steel and Aluminum Alloy pipe may be used in the unshaded area.

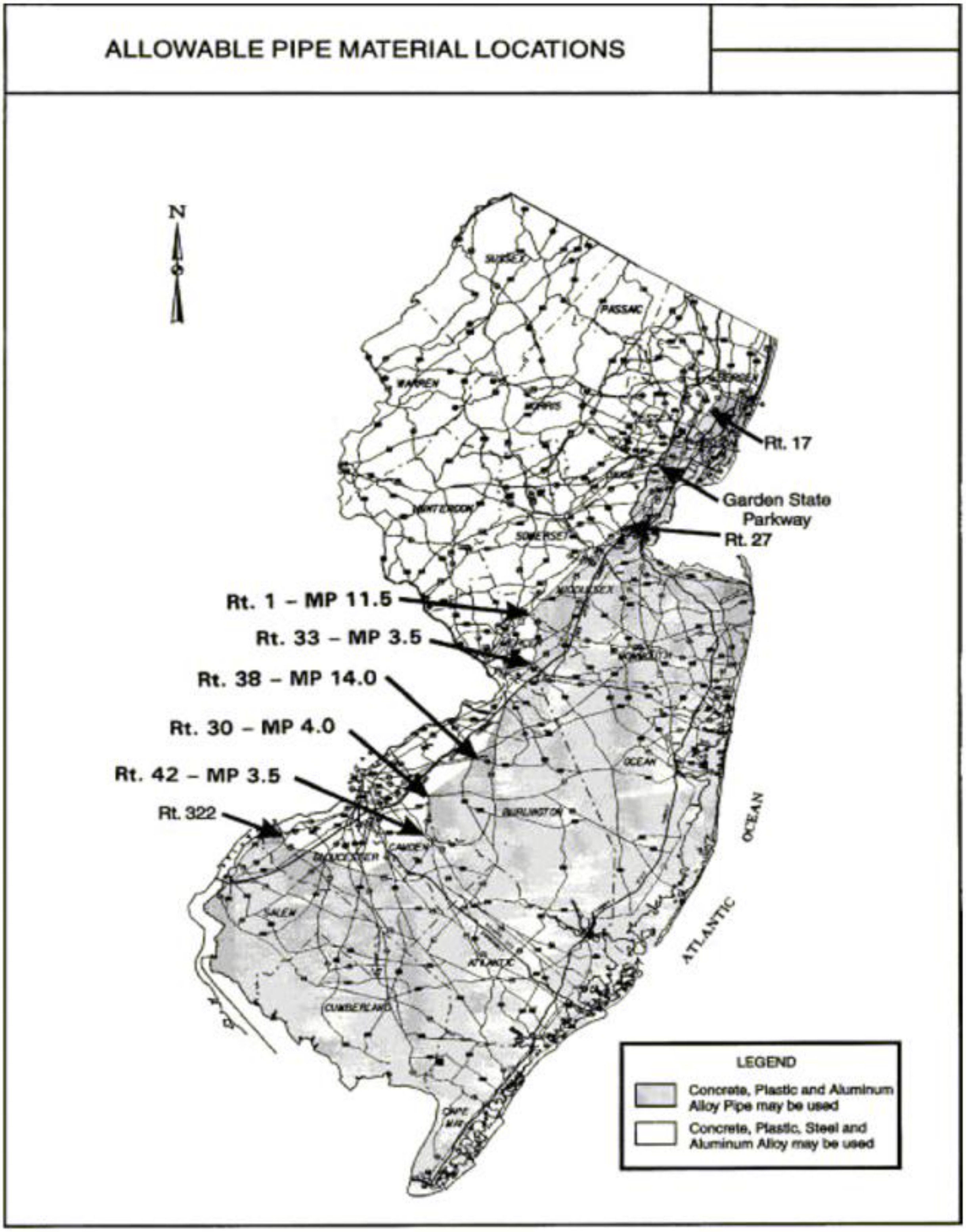
N. Alternate Items:

   a. When the estimated cost of the pipes is more than $50,000, alternate bid items are required.
   b. Alternate pipe materials include corrugated metal, aluminum alloy, and HDPE.
   c. Some materials may be eliminated as alternate items due to unstable support, high impact, concentrated loading, limited clearance, steep gradients, etc.

O. The drainage layout should attempt to avoid conflicts with existing underground utilities and such items as utility poles, signal pole foundations, guide rail posts, etc. Implementation of the following design approaches may be necessary.

   a. Use of pipe material with the lowest friction factor to minimize pipe size
   b. Use of elliptical or arch pipe to minimize vertical dimension of pipe.
   c. Test pits should be obtained early in the design process to obtain horizontal and vertical information for existing utilities. If the suggested design approaches do not avoid conflict, use of special drainage structures may be used to avoid the utility.

Figure 6-1


ALLOWABLE PIPE MATERIAL LOCATIONS
N
SUSSEX
PASSAIC
WARREN
MORRIS
HUNTERDON
SOMERSET
MIDDLESEX
MONMOUTH
OCEAN
BURLINGTON
CAMDEN
GLOUCESTER
SALEM
ATLANTIC
CUMBERLAND
CAPE MAY
ATLANTIC OCEAN
Rt. 17
Garden State Parkway
Rt. 27
Rt. 1 – MP 11.5
Rt. 33 – MP 3.5
Rt. 38 – MP 14.0
Rt. 30 – MP 4.0
Rt. 42 – MP 3.5
Rt. 322
LEGEND
Concrete, Plastic and Aluminum Alloy Pipe may be used
Concrete, Plastic, Steel and Aluminum Alloy may be used

When alternate bid pipe materials are required, separate hydraulic calculations must be developed and submitted for each material considered (concrete, corrugated metal/aluminum alloy) using the respective roughness coefficients. The reason for exclusive use of a pipe material must be explained in the Drainage Report.

P. Round corrugated metal pipe shall have helical corrugations, except that annular corrugated pipe may be used where velocity reduction is desired.
Q. Drainage structures must accommodate all pipe materials used including concrete, corrugated metal, aluminum alloy, and HDPE.
R. Aluminum alloy pipe shall not be used as a section or extension of a steel pipe.
S. Precast manholes or inlets shall not be used for pipes 54 inches or larger diameter or when three or more pipes tie in and at least two of them are connected at some angles. When these conditions exist, cast-in-place inlets or manholes are more practical.
T. Cleaning existing drainage pipes and structures shall be incorporated on all projects when the existing drainage system has substantial accumulation of sediments. The cleaning shall extend to the first structure beyond the project limits.
U. On projects where contaminated areas have been identified, the drainage system should be designed to avoid these locations, if possible. If avoidance is not feasible, a completely watertight conveyance system, including structures such as manholes, inlets, and junction chambers, should be designed to prevent contaminated groundwater or other pollutants from entering the system. Possible methods to accomplish this include joining pipe sections with a watertight sealant and/or gaskets, or the use of welded steel pipe. Retrofitting existing pipes to make them watertight may require installation of an appropriate internal liner. The design engineer shall provide recommendations prior to proceeding with the final design.
V. The soffits (overts) between the inflow and outflow pipes at a drainage structure shall be matched where possible. A minimum 1 inch drop between inverts within the structure shall be provided, if feasible.
W. Existing drainage facilities that are not to be incorporated into the proposed drainage system are to be completely removed if they are in conflict with any element of the proposed construction. Existing drainage facilities that are not to be incorporated into the proposed drainage system that do not conflict with any element of the proposed construction are to be abandoned. Abandonment of existing drainage facilities requires the following:

    1. Plugging the ends of the concrete pipes to remain. Metal pipes shall be either removed or filled.
    2. Filling abandoned pipes in accordance with geotechnical recommendations.
    3. Removing the top of the drainage structure to 1 foot below the bottom of the pavement box, breaking the floor of the structure, and filling the structure with either granular material or concrete in accordance with geotechnical recommendations.

Y. A concrete collar, as shown in the standard detail CD-602-1.3, will be used to join existing to proposed pipe of similar materials unless an approved adapter fitting is available.

6.3 Storm Sewer Design

Hydraulic design of the drainage system is performed after the locations of inlets, storm drain layout, and outfall discharge points have been determined. Hydraulic design of the drainage pipe is a two step process. The first step establishes the preliminary pipe size based on hydrology and simplified hydraulic computations. The second step is the computation of the hydraulic grade line (HGL) for the system. This step refines the preliminary pipe size based on calculation of the hydraulic losses in the system using the hydrology computed in the first step for each section of pipe. The procedures to be performed in step 1 are presented in Section 6.4, "Preliminary Pipe Size". The procedures to be performed in step 2 are presented in Section 6.5, "Hydraulic Grade Line Computations".

6.4 Preliminary Pipe Size

The preliminary design proceeds from the upstream end of the system toward the outlet at which the system connects to the receiving downstream system. The design runoff for each section of pipe is computed by the Rational formula using the total area that contributes runoff to the system and the Time of Concentration to the upstream end of the pipe. The Time of Concentration increases in the downstream direction of the design and the rainfall intensity consequently decreases. All runoff from the contributing area is assumed to be captured. The inlet capture and by-pass computations used to determine the inlet layout are not used in the hydraulic computation.

The preliminary storm drain size should be computed based on the assumption that the pipe will flow full or practically full for the design runoff. The Manning equation should be used to compute the required pipe size. This preliminary procedure determines the required pipe size based on the friction losses in the pipe. All other losses are disregarded in the preliminary design. In general, the longitudinal grade of the roadway over the pipe being designed should be used as the slope in the hydraulic computation where practical. The HGL computations, as explained in Section 6.5, consider all losses and establish the actual pipe size required.
Figure 6-2 is recommended for use as guidance in performing the preliminary drainage system design. Use of computer programs to perform the computations is encouraged. The computational procedures and output results and presentation format presented in the FHWA Hydrain-Hydra program are recommended for use. Use of other computer programs is acceptable provided, as a minimum, the computational procedures and presentation of output are similar to those presented in Figure 6-2.

The following is an explanation of the Preliminary Storm Drain Computation Form, Figure 6-2. Data is to be presented for each reach of pipe being designed. The numbers refer to each column in Figure 6-2.

PRELIMINARY STORM DRAIN COMPUTATION FORM	FIGURE 6-2

Computed: ____________ **Date:** ____________ **Route:** ______________________________

Section: ______________________________

Checked: ____________ **Date:** ____________ **County:** ______________________________

Station and Offset (1)		L (ft)	Drainage Area "A" (Acres)		Runoff Coef-ficient "C"	"A" x "C"		Flow Time "Tc" (min.)			Rainfall "I" in/hr	Total **Chapter** unoff Q=CIA ft^3/S	Dia. Pipe ft	Slope ft/ft	Capacity Full ft^3/S	Velocity ft/s		Invert Elevation	
Junction From	Junction To	(2)	Incre-ment (3)	Total (4)	(5)	Incre-ment (6)	Total (7)	Overland To Inlet (8)	In U/S Pipe (9)	Cum. Total in Pipe* (10)	(11)	(12)	(13)	(14)	(15)	Flowing Full (16)	Design Flow (17)	U/S End (18)	D/S End (19)

- For Time of Concentration, use larger of overland flow to inlet or cumulative time in pipe.

1. **Station and Offset**
   Input the location of the upstream and downstream structure for each pipe reach being designed referenced from the base line, survey line, or profile grade line (PGL) shown on the construction documents.
2. **Length in feet**
   Input the distance between the centerline of the upstream and downstream structure.
3. **Incremental Drainage Area in acres**
   Input the drainage area to each structure for each area with a different runoff coefficient that contributes runoff to the upstream structure.
4. **Total Drainage Area in acres**
   Input the cumulative total drainage area. This is a running total of column 3.
5. **Runoff Coefficient**
   Input the rational method runoff coefficient for each area contributing runoff to the structure.
6. **Incremental "A" x "C"**
   Input the incremental drainage area times its runoff coefficient for each area contributing runoff to the structure.
7. **Total "A" x "C"**
   Input the cumulative drainage area times the runoff coefficient. This is a running total of column 6.
8. **Flow Time (Time of Concentration) to Inlet in Minutes**
   Input the overland Time of Concentration to each structure.
9. **Flow Time in Pipe in Minutes**
   Input the flow time in the pipe upstream of the upstream junction (junction from). This time is computed by dividing the pipe length by the actual design flow velocity in the pipe (Column #2 divided by Column #17) for the pipe section upstream of the junction from structure (Column #1). The first pipe length will have no value. The flow time in the pipe will be used to compute the cumulative Time of Concentration (travel time) in the pipe.
10. **Cumulative Time in the Pipe in Minutes**
    Input the cumulative time in the pipe. This is a running total of column 9. If the overland flow to the inlet is greater than the cumulative time in the pipe, then that overland flow time will be added to subsequent flow time in the pipe to determine the longest cumulative Time of Concentration.
11. **Rainfall Intensity "I" in inches per Hour**
    Input the rainfall intensity using Figures 3-1 through 3-4 and the longest Time of Concentration. The longest Time of Concentration is determined by using the larger of the overland flow time to the inlet (column 8) or the cumulative time in the pipe (column 10).
12. **Total Runoff (Q = CIA) in cubic feet per Second**
    Compute the total runoff using the area, runoff coefficient, and rainfall intensity identified in step 11.
13. **Pipe Diameter in feet**
    Compute the required pipe diameter using Manning's equation based on full flow. The tailwater is assumed to be at the elevation of the pipe soffit.
14. **Slope in feet per feet**
    Input the pipe slope used for the pipe design. The slope is typically as close as possible to the roadway longitudinal grade over the pipe reach being designed.
15. **Capacity in cubic feet per Second**
    Compute the pipe capacity using the Manning's equation and full flow conditions.

16. **Velocity (full) in feet per Second**
    Compute the pipe velocity using the full pipe capacity (V = Q/A).
17. **Velocity (design) in feet per Second**
    Compute the pipe velocity using the design discharge.
18. **Invert Elevation (Upstream End)**
    Input the pipe invert elevation at the upstream end.
19. **Invert Elevation (Downstream End)**
    Input the pipe invert elevation at the downstream end.

6.5 Hydraulic Grade Line computations

The Hydraulic Grade Line (HGL) should be computed to determine the water surface elevation throughout the drainage system for the design condition. The HGL is a line coinciding with either (1) the level of flowing water at any point along an open channel, or (2) the level to which water would rise in a vertical tube connected at any point along a pipe or closed conduit flowing under pressure. The HGL is normally computed at all junctions, such as inlets and manholes. All head losses in the storm drainage system are considered in the computation. The computed HGL for the design runoff must remain at least 1 foot below the top of grate or rim elevation.

Hydraulic control, also commonly referred to as "tailwater", is the water surface elevation from which the HGL calculations are begun. "Tailwater" elevation is established by determining water surface elevation at the locations where the new drainage system will discharge to the receiving waterway, such as a stream, ditch, channel, pond, lake, or an existing or proposed storm sewer system. The tailwater selected for the design should be the water surface elevation in the receiving waterway at the Time of Concentration for the connecting roadway storm sewer being designed or analyzed.

When the system is under pressure and when a higher level of accuracy is required considering storage in the pipe system, pressure flow routing can be performed using computer programs such as the "Pressure Flow Simulation" option in the FHWA Hydrain-Hydra program. Use of a pressure flow routing in the design of a new drainage system or analysis of an existing drainage system should be evaluated early in the initial design. A pressure flow routing is typically appropriate only in special cases, primarily when the available storage attenuates the peak discharge to the extent that downstream pipe sizes are minimized.

Figures 6-3 and 6-4 are recommended for use as guidance in performing HGL computations. HGL line computations must be provided for all projects. Use of computer software acceptable to the Department to perform the computational procedures is encouraged. The computational procedures, output results, and presentation format similar to what is presented in Figures 6-3 and 6-4 are required as a minimum.

The following is an explanation of the computation of the Hydraulic Grade Line using Figure 6-3. The computed hydraulic grade line (HGL) for the design runoff must remain at least 1 foot below the roadway finished grade elevation at the drainage structure. Data is to be presented for each reach of pipe being designed. The pipe designation presented in the explanation refers to the pipe being designed unless otherwise noted. The numbers refer to each column in Figure 6-3.

1. **Station and Offset**
   Input the location of the upstream and downstream structure for each pipe reach being designed, referenced from the base line, survey line, or profile grade line (PGL) where applicable from the construction documents.
2. **Pipe Diameter (Ø) in feet**
   Input downstream pipe diameter.
3. **Flow (Q) in cubic feet per Second**
   Input flow in downstream pipe (outflow pipe).
4. **Pipe velocity in feet per Second**
   Input the design velocity of the pipe.
5. **Hydraulic Radius (R) in feet**
   Input the hydraulic radius (area divided by wetted perimeter) of the pipe.
6. **Length (L) of Pipe in feet**
   Input the distance between the centerline of the upstream and downstream structure.
7. **Manning's "n" Roughness Coefficient**
   Input the Manning's coefficient "n". Use 0.012 for concrete and smooth interior plastic pipe. The Manning's "n" values for corrugated metal and aluminum alloy pipe are shown in Table 6-2.
8. **Velocity Head (h) in feet**
   Compute the velocity head, $h = V^2/2g$, Where g = acceleration due to gravity.
9. **Friction Loss (H_f) in feet**
   Compute the friction loss in the pipe using the equation:

$$H_f = \frac{29.14 n^2 L}{R^{1.33}} \times \frac{V^2}{2g}$$

HYDRAULIC GRADE LINE COMPUTATION FORM	**FIGURE 6-3**

Computed: ____________ **Date:** ____________ **Route:** ______________________________

Section: ______________________________

Checked: ____________ **Date:** ____________ **County:** ______________________________

Station & Offset (1)		(2)	Q (3)	V (4)	R (5)	L (6)	n (7)	h (8)	H_f (9)	H_e (10)	H_i (11)	H_s (12)	H_t (13)	TW (14)	HGL (15)	TOS (16)	CL (17)
Junction From	Junction To	Dia. ft	Flow ft^3/S	Vel. ft/s	Hydraulic radius ft	Length ft	Manning's	Vel. Head ft	Fric. Loss ft	Exit Loss ft	Entr. Loss ft	Struct. Loss* ft	Total Head Loss ft	Tail-water Elev. ft	Head-water Elev. ft	Top of Struct. Elev. ft	TOS-HGL ft

h = Velocity head, = $(V)^2/2g$

H_i = Entrance Loss = $K_i(V)^2/2g$

Refer to Table 6-3 for values of K_i

H_f = Friction Loss, = $\dfrac{29.14N^2L}{R^{1.33}} \times \dfrac{(V)^2}{2g}$

H_e = Exit Loss, $H_e = (V)^2/2g$

* For structural (junction) losses in inlets, manholes, see Figure 6-3.

STRUCTURAL AND BEND LOSS COMPUTATION FORM	**FIGURE 6-4**

Computed: ____________ **Date:** ____________ **Route:** ________________________________

Section: ________________________________

Checked: ____________ **Date:** ____________ **County:** ________________________________

(1)	(2)	Q (3)	v (4)	$\frac{v^2}{2g}$ (5)	(6)	(7)	K_s (8)	H_s (9)	A (10)	K_b (11)	H_b (12)	$H_s + H_b$ (13)
Junction Station & Offset	Downstream Dia. ft	Downstream Flow ft^3/S	Downstream Velocity ft/s	Velocity Head ft	Junction Type (L,N or O)	Flow Type (P or O)	Structural Loss Coeff.	Structural Loss ft	Angle deg.	Bend Factor	Bend Loss ft	Structural Loss + Bend Loss ft

H_s = Structural Loss = $K_S \times \frac{(V)^2}{2g}$, K_s from Table 6-4

H_b = Bend Loss = $K_b \times \frac{(V)^2}{2g}$, K_b from Figure 7-1

NOTES:

1) Junction Type
   L = with Lateral
   N = with No Lateral
   O = with Opposed Laterals

2) Flow Type
   P = Pressure
   O = Open Channel

Table 6-3
Entrance Loss Coefficients (K_i)

This table shows values of the coefficient Ki to apply to the velocity head $V^2/2g$ to determine the loss of head at the entrance of a structure such as a culvert or conduit, operating full or partly full with control at the outlet.

Entrance head loss $H_i = K_i V^2/2g$

Type of Structure and Design of Entrance	**Coefficient, K_i**
A. Concrete Pipe	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls Socket end of pipe (groove-end) Square-edge Rounded (radius = D/12)	 0.2 0.5 0.2
Mitered to conform to fill slope	0.7
End-section conforming to fill slope *	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side or slope-tapered inlet	0.2
B. CMP or CMPA	
Projecting from fill (no headwalls)	0.9
Headwall or headwall and wingwalls Square-edge	 0.5
Mitered to conform to fill slope	0.7
End-section conforming to fill slope *	0.5
C. Concrete Box	
Headwall parallel to embankment (no wingwalls) Square-edged on 3 edges Rounded on 3 edges to radius of 1/12 barrel dimension, Or beveled edges on 3 sides	 0.5 0.2
Wingwalls at 30 - 75 degrees to barrel Square-edged at crown Crown edge rounded to radius of 1/12 barrel dimension, Or beveled top edge	 0.4 0.2
Wingwalls at 10 - 25 degrees to barrel Square-edged at crown	 0.5
Wingwalls parallel (extension of sides) Square-edged at crown	 0.7

***NOTE**: "End sections conforming to fill slope", made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control.

10. **Exit Loss (H_e) in feet**

Compute the exit loss of the drainage system using the equation:

$\mathbf{H_e} = \mathbf{V^2/2g}$, Where **V** = velocity of outflow pipe

The exit loss is computed where the drainage system discharges to a swale, stream, pond, etc. via a headwall or a pipe open end. This loss is calculated for the last downstream pipe segment at the outlet end of the pipe being designed.

11. **Entrance Loss (H_i) in feet**
    Compute the entrance loss of the drainage system using the equation:

    $H_i = K_i V^2/2g$,
    Where
    K_i = Entrance Loss Coefficient

    The entrance loss is computed at the upstream end of the system where the flow enters the first structure. This is either at a headwall/ end section or the pipe in the beginning upstream inlet. Entrance loss coefficients are presented in Table 6-3.
12. **Structural Loss (H_s) in feet**
    Input the structural loss from Figure 6-4. The structural loss corresponds to the structure at the upstream end of the pipe segment or "junction from".
13. **Total Head Loss (H_t) in feet**
    Compute the total head loss by adding the exit, entrance, friction, and structural loss. The exit and entrance losses are only added at the beginning and end of the pipe system, respectively.
14. **Tailwater Elevation (TW) in feet**
    Input the tailwater elevation at the downstream end of the pipe segment being designed. For the last downstream pipe segment, the tailwater elevation is established by determining the water surface elevation at the location where the pipe discharges to a stream, ditch, channel, pond, lake, or an existing or proposed storm sewer system. The tailwater selected for the design should be the water surface elevation in the receiving waterway at the Time of Concentration for the connecting roadway storm sewer being designed or analyzed. The tailwater elevation for each upstream pipe segment will be the computed headwater elevation (HGL) for the downstream pipe segment.
15. **Headwater Elevation (HGL) in feet**
    Compute the HGL at the upstream end of the pipe segment by adding the total head loss (H_t) to the tailwater elevation (TW) at the downstream end of the pipe.
16. **Top of Structure (TOS) Elevation in feet**
    Input the top of structure elevation which is the top of grate for inlets and rim elevation for manholes.
17. **Clearance (CL) in feet**
    Compute the clearance or difference in elevation between the top of structure (TOS) and the headwater elevation (HGL). The HGL shall be a minimum of 1 foot below the TOS.

The following is an explanation of the computation of structural losses using Figure 6-4. Data is to be presented for each reach of pipe being designed. The numbers refer to each column in Figure 6-4.

1. **Station and Offset**
   Input the location of each drainage structure referenced from the base line, survey line, or profile grade line (PGL) where applicable from the construction documents.
2. **Pipe Diameter (Ø) in feet**
   Input downstream pipe diameter (outflow). Equivalent diameter for elliptical or arch pipes may be used.
3. **Flow (Q) in cubic feet per second**
   Input flow in downstream pipe (outflow pipe).
4. **Downstream Velocity (v) in feet per second**

Input the velocity in the pipe.

5. **Velocity Head (h) in feet**
   Compute the velocity head, $h=V^2/2g$
6. **Structure Lateral Configuration**
   The structural loss coefficient is related to the structure lateral configuration and type of flow. The lateral configuration designation is as follows:

   L = Junction with lateral
   N = Junction with no lateral
   O = Junction with opposed laterals

7. **Flow Type**
   The structural loss coefficient is related to the structure lateral configuration and type of flow. The flow type designation is as follows:

   P = Pressure flow
   O = Open channel flow

8. **Structural Head Loss Coefficient**
   The structural head loss coefficient is related to the structure lateral configuration and type of flow. Insert the coefficient selected from Table 6-4:

Table 6-4
Structure Head Loss Coefficient (K_s)

Flow Condition	Lateral Configuration	Coefficient
Open Channel	90° Lateral	0.2
Open Channel	No Lateral	0.0
Open Channel	Opposed	0.2
Pressure	90° Lateral	1.0
Pressure	No Lateral	0.3
Pressure	Opposed	1.0

   Proper application of the structural loss to the drainage system requires an understanding of which pipe(s) is (are) considered the lateral(s) and which pipes are considered the main. For simplicity, the inflow pipe with the majority of the flow entering the structure is considered the main. All other inflow pipes are considered laterals.

   The hydraulic grade line computation for each lateral begins with the water surface elevation for the junction, which includes the structural head loss and bend head loss for the structure. No other losses are associated with the connection of the lateral to the junction.

9. **Structural Loss in feet**
   Compute the structural loss as the product of the structural loss coefficient (column 8) and velocity head (column 5).
10. **Angle (A) in degrees**
    Input the deflection angle between the inflow and outflow main pipes. The angle should be between 0 and 90 degrees.
11. **Bend Factor**
    Insert bend factor from Figure 7-1.
12. **Bend Loss in feet**

Compute the bend loss as the product of the bend factor (column 11) and velocity head (column 5).

13. **Structural Loss + Bend Loss in feet**
Compute the sum of the structural loss (column 9) and the bend loss (column 12).

7.0 Median Drainage

7.1 Introduction

The basic purpose of a median is to separate opposing lanes of traffic. The widths, grade and shape of a median is determined for the most part by safety considerations. A wide, shallow, depressed median is usually selected as best fulfilling the median purpose.

A provision to drain the median by means of inlets must be included in the median design. Median inlets shall be provided to limit the depth of flow to 6 inches to confine the spread to the median and below the pavement subgrade. This section contains procedures and criteria for the design of median drainage.

7.2 Median Inlet Type

All median inlets are to be Type "E".

7.3 Median Design Criteria - Continuous Grade

Median inlets should intercept the total design flow from its discharge area plus any by-pass from upstream. The drainage area to each inlet must be adjusted by inlet spacing to limit the design flow to a maximum depth of 6 inches. Because of the variable parameters in the spread calculations, each inlet must be investigated.

A. The recurrence interval used in the design is the same as that of the longitudinal roadway system.

7.4 Procedure for Spacing Median Drains

Channel capacity shall be computed using the procedures presented in Section 4.0, Channel Design.

Inlet capture for inlets on grade shall be computed using the weir equation stated as follows:

$$\mathbf{Q_i = C_w P y^{1.5}}$$

where

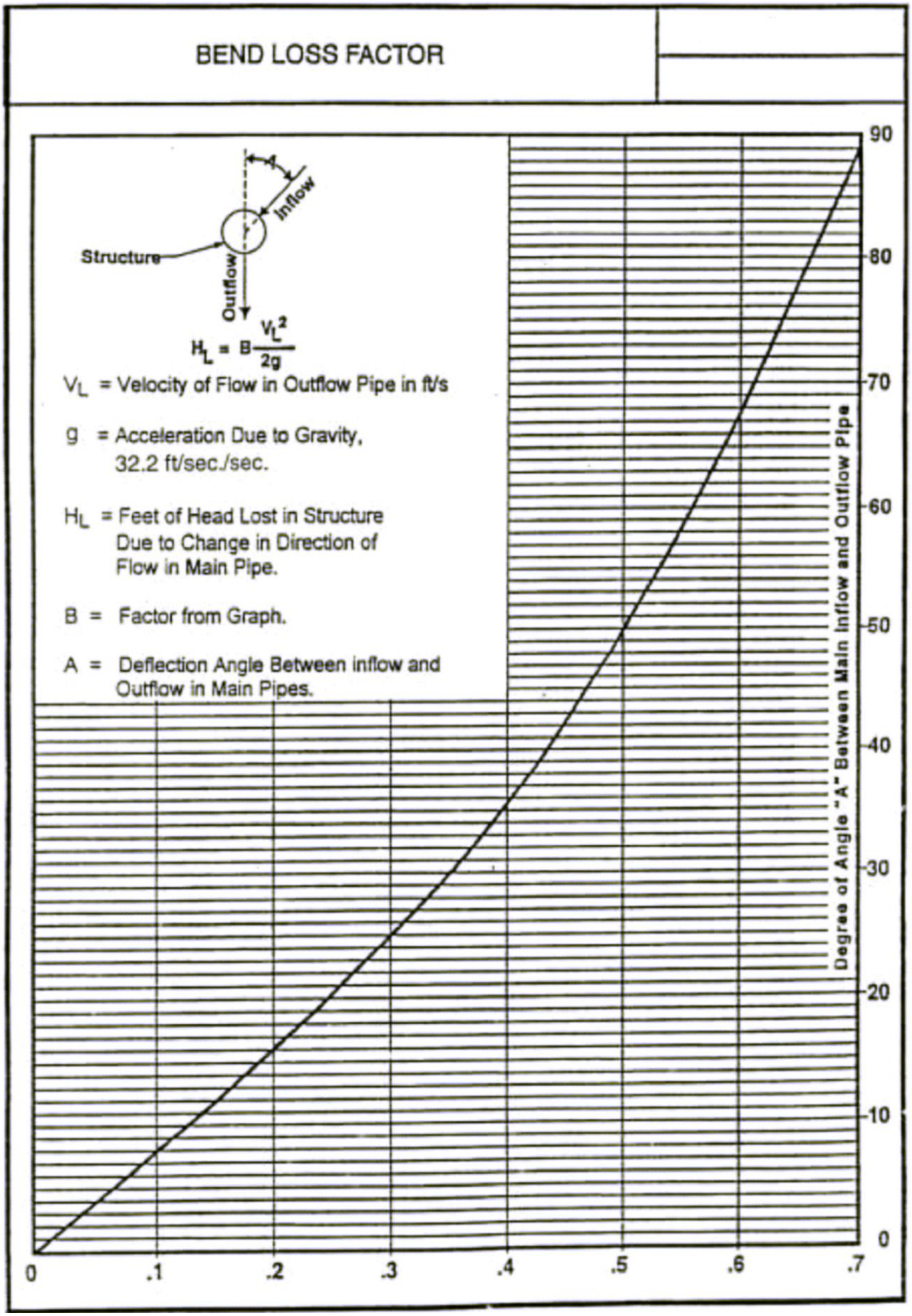
$\mathbf{Q_i}$ = flow rate intercepted by the grate ft^3/s

$\mathbf{C_w}$ = weir coefficient

$\mathbf{P}$ = weir length (ft)

$\mathbf{y}$ = depth (ft) for the approach flow

Figure 7-1


BEND LOSS FACTOR
A
Inflow
Structure
Outflow
$H_L = B\frac{V_L^2}{2g}$
V_L = Velocity of Flow in Outflow Pipe in ft/s
g = Acceleration Due to Gravity, 32.2 ft/sec./sec.
H_L = Feet of Head Lost in Structure Due to Change in Direction of Flow in Main Pipe.
B = Factor from Graph.
A = Deflection Angle Between inflow and Outflow in Main Pipes.
Degree of Angle "A" Between Main Inflow and Outflow Pipe
90
80
70
60
50
40
30
20
10
0
0
.1
.2
.3
.4
.5
.6
.7

The weir flow coefficient is 3.0. The weir length to be used is the frontal flow length of the inlet.

Inlet capture for inlets at low points shall be computed using the procedures in Section 5.8 "Capacity of Grate Inlets at Low Points".

Judgment should be used in a cut section to place these inlets economically as well as functionally. Some leeway is afforded the design engineer to place the median inlets opposite roadway edge inlets. This simplifies connections and reduces pipe lengths. The water that bypasses the inlet because of the above, should be added to the next inlet's design runoff.

8.0 Culvert Design

8.1 Introduction

A highway embankment constitutes a barrier to the flow of water where the highway crosses water courses. A culvert is a closed conduit that provides a means of carrying the flow of water through the embankment.

8.2 Culvert Types

A. **Pipes**: Metal and reinforced concrete pipe culverts are shop manufactured products available in a range of sizes in the standard shapes. Metal pipes (aluminum and steel) are available in round and arch shapes. Reinforced Concrete pipes are available in round and elliptical shapes. Round shapes are generally more economical, due to their greater strength.

   Pipe flow characteristics for different pipes change due to their relative roughness.

   Additional capacity can be obtained with multiple pipe installations. Multiple installations are accomplished by installing several individual culvert pipes parallel to each other with enough separation to allow for proper compaction.

B. **Reinforced Concrete Boxes (RCB's)**: Box culverts are either precast off-site or constructed in the field by forming and pouring. Box culverts may be constructed to any desired size in either square or rectangular shapes. These designs may be easily altered to allow for site conditions. The flow characteristics of RCB's are very good as their barrels provide smooth flow and their inlet may be designed for extra efficiency where needed.

   Where a multiple culvert installation is indicated, the RCB may be constructed with two or more barrels. Stream Encroachment Permit requirements may dictate when multiple culverts can be used. The minimum width, if possible, will be 10 feet per box. For streams with a drainage area greater than 50 acres, the Stream Encroachment Permit requirements will also dictate the need to provide a fish passage in at least one box culvert. Guidance regarding fish passage provisions in culverts are presented in Section 8.8.

8.3 Culvert Location

The alignment of a culvert in both plan and profile should ensure efficient hydraulic performance, as well as keep the potential for erosion and sedimentation to a minimum. The criteria given in Section 4.0, "Channel Design", should be considered in the location of the culvert. Usually, the ideal location for the culvert is the existing channel, with the slope the same as the existing channel.

8.4 Culvert Selection

Select a culvert type and size that is compatible with hydraulic performance, structural integrity and economics. The structural requirements for various pipes may be found in references (1), (2), and (3).

8.5 Culvert Hydraulics

Laboratory tests and field observations show two major types of culvert flow: flow with inlet control and flow with outlet control. Different factors and formulas are used to compute the hydraulic capacity of a culvert for each type of control. Under inlet control, the cross-sectional area of the culvert barrel, the inlet geometry and the amount of headwater or ponding at the entrance are of primary importance. Outlet control involves the additional consideration of the elevation of the tailwater in the outlet channel and the slope, roughness and length of the culvert barrel.

It is possible by involved hydraulic computations to determine the probable type of flow under which a culvert will operate for a given set of conditions. The need for making these computations may be avoided, however, by computing headwater depths from available charts and/or computer programs for both inlet control and outlet control and then using the higher value to indicate the type of control and to determine the headwater depth. This method of determining the type of control is accurate except for a few cases where the headwater is approximately the same for both types of control. Refer to FHWA HDS-5 - Hydraulic Design of Highway Culverts for detailed culvert design procedures.

8.6 Culvert End Structures

Culvert end structures may be used for the following purposes:

A. To improve the hydraulic efficiency of the culvert.
B. To provide erosion protection and prevent flotation.
C. To retain the fill adjacent to the culvert.

These structures include headwalls, concrete flared end sections, corrugated metal end sections, and improved inlet structures to increase capacity. Each type is described in the following narrative.

A. **Headwall**: A headwall is a retaining wall attached to the end of a culvert. (see current Standard Construction Details CD-610-1). The alignment of the headwall should be normal to the centerline of the barrel to direct the flow into the barrel. The wingwalls should be long enough to prevent spillage of the embankment into the channel. A cutoff wall attached to the downstream end of the unit if a concrete apron is not provided at the headwall. The cutoff wall may be a concrete unit across the entire width of the downstream end of the flared end section. The cutoff wall shall be a minimum of 1.5 feet thick and 3.0 feet deep (see current Standard Construction Details).
B. **Concrete Flared End Sections**: A concrete flared end section is a precast unit with a beveled and flared end that provides an apron at the outlet end of the pipe. The bevel approximately conforms to embankment slope. Limited grading of the embankment is usually required around the end of the flared end section. Installation of a flared end section requires installation of a cutoff wall attached to the downstream end of the unit. The cutoff wall may be a concrete unit across the entire width of the downstream end of the flared end section. The cutoff wall shall be a

minimum of 1.5 feet thick and 3.0 feet deep (see current Standard Construction Details).

C. **Corrugated Metal End Sections**: A corrugated metal end section is a beveled and flared end that provides an apron at the outlet end of the pipe. The bevel approximately conforms to embankment slope. Limited grading of the embankment is usually required around the end of the end section. Installation of an end section requires installation of a cutoff wall attached to the downstream end of the unit. The cutoff wall may be a concrete unit across the entire width of the downstream end of the section. The cutoff wall shall be a minimum of 1.5 feet thick and 3.0 feet deep (see current Standard Construction Details).

D. **Improved Inlet**: An improved culvert inlet incorporates inlet geometry refinements to increase the capacity of a culvert operating with inlet control. These geometry improvements include beveled edges, side tapers and slope tapers functioning either individually or in combination.

8.7 Flood Routing at Culverts

The presence of substantial storage volume below the allowable headwater elevation at the upstream end of a culvert warrants evaluation of the resultant peak flow attenuation. The reduced peak discharge resulting from attenuation yields a reduced culvert size for a new crossing. Attenuation of the peak discharge at existing crossings may indicate that the existing culvert is adequate or may reduce the size of the relief or replacement culvert. For this reason, flood routing computations shall be performed for all culvert locations except where the proposed topography indicates that limited storage volume, such as is typical with deep incised channels, is available.

Flood routing evaluation at a culvert provides a realistic indication of hydrologic conditions at the culvert entrance. A more realistic assessment can be made where environmental concerns are important. The extent and duration of temporary upstream ponding determined by the flood routing computations can help improve the environmental assessment of the proposed construction.

The design procedure for flood routing through a culvert is the same as for reservoir routing. Additional information on flood routing and storage is included in Section 3.6.

8.8 Fish Passage

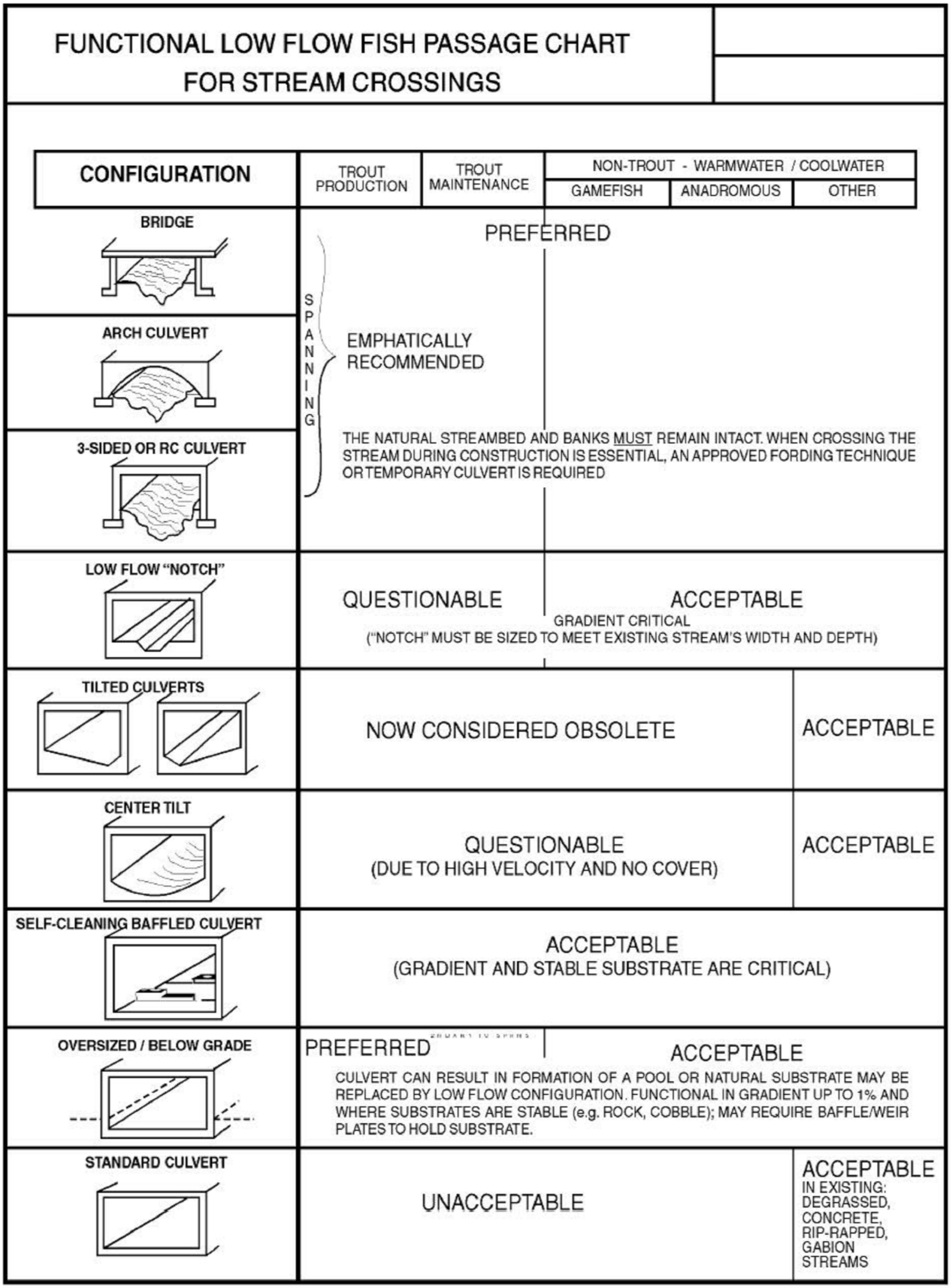
Fish passage is historically a concern with culverts. Failure to consider fish passage may block or impede upstream fish movements in the following ways:

- Outlet of the culvert is installed above the streambed elevation to where fish may not be able to enter.
- Scour lowers the streambed downstream of the culvert outfall and the resulting dropoff creates a potential vertical barrier.
- High outlet velocity may provide a barrier.
- Higher uniform velocities within the culvert than occur in the natural channel may prevent fish from entering or transiting the culvert.
- Abrupt drawdown, turbulence, and accelerated flow at the inlet to the culvert entrance may prevent fish from exiting the culvert.
- Natural channel replaced by an artificial channel may have no zones of quiescent water in which fish can rest.

- Debris barriers (including ice) upstream or within the culvert may stop fish movement.
- Shallow depths within the culvert during minimum flow periods may preclude fish passage.

The design engineer is encouraged to refer to the NJDEP Technical Manual for Land Use Regulation Program, Bureau of Inland and Coastal Regulations, Stream Encroachment Permits, in addition to Figure 8-1 for the latest acceptable methods for providing fish passage in all proposed box culvert installations. For more guidance on fish passage provisions in proposed culvert installations, contact the NJDEP Division of Fish and Wildlife.

Figure 8-1



NOTE: (1) TWIN OR MULTICELL CULVERTS SHOULD HAVE THE LOW FLOW TREATMENT IN A SINGLE CELL.
(2) NATURAL SUBSTRATE AND BAFFLES SHAPING A LOW FLOW CONFIGURATION CAN CREATE A "LOTIC ECO-CULVERT"

Source: New Jersey Division of Fish, Game and Wildlife

9.0 Conduit Outlet Protection

The purpose of conduit outlet protection is to provide a stable section of area in which the exit velocity from the pipe is reduced to a velocity consistent with the stable condition downstream. The need for conduit outlet protection shall be evaluated at any location where drainage discharges to the ground surface or a channel, ditch or stream. This may occur at the downstream end of culverts or other drainage systems.

The need for conduit outlet protection shall be determined by comparing the allowable velocity for the soil onto which the pipe discharges to the velocity exiting the pipe. The allowable velocity for the soil shall be that given in the NJDOT Soil Erosion and Sediment Control Standards Manual. The velocity in the pipe shall be that which occurs during passage of the design storm or of the 25-year storm, whichever is greater. When the velocity in the pipe exceeds the allowable velocity for the soil, outlet protection will be required.

For a detail of conduit outlet protection for a flared end section or headwall, see the Standard Roadway Construction Detail CD-602-1.4, "Stormwater Outfall Protection".

9.1 Riprap Size and Apron Dimensions

Conduit outlet protection and apron dimensions shall be designed in accordance with procedures in the NJDOT Soil Erosion and Sediment Control Standards Manual. The minimum d_{50} stone size shall be 6 inches. A tail water depth equal to 0.2 D_o shall be used where there is no defined downstream channel or where T_w cannot be computed.

9.2 Energy Dissipators

Energy dissipators are typically required when the outlet velocity is 15 ft/s or greater. Energy dissipators shall be provided when the stable velocity of the existing channel is exceeded, or when design of standard riprap conduit outlet or channel protection results in an impractical stone size and/or thickness. Energy dissipators for channel flow have been investigated in the laboratory, and many have been constructed, especially in irrigation channels. Designs for highway use have been developed and constructed at culvert outlets. All energy dissipators add to the cost of a culvert; therefore, they should be used only to prevent or to correct a serious erosion problem that cannot be corrected by normal design of standard soil erosion and sediment control elements.

The judgment of engineers is required to determine the need for energy dissipators at culvert outlets. As an aid in evaluating this need, culvert outlet velocities should be computed. These computed velocities can be compared with outlet velocities of alternate culvert designs, existing culverts in the area, or the natural stream velocities. In many streams the maximum velocity in the main channel is considerably higher than the mean velocity for the whole channel cross section. Culvert outlet velocities should be compared with maximum stream velocities in determining the need for channel protection. A change in size of culvert does not change outlet velocities appreciably in most cases.

Outlet velocities for culverts flowing with inlet control may be approximated by computing the mean velocity for the culvert cross section using Manning's equation.

Since the depth of flow is not known, the use of tables or charts is recommended in solving this equation. The outlet velocity as computed by this method will usually be high because the normal depth, assumed in using Manning's equation, is seldom reached in the relatively short length of the average culvert. Also, the shape of the outlet channel, including aprons and wingwalls, has much to do with changing the velocity occurring at the end of the culvert barrel. Tailwater is not considered effective in reducing outlet velocities for most inlet control conditions.

In outlet control, the average outlet velocity will be the discharge divided by the cross-sectional area of flow at the outlets. This flow area can be either that corresponding to critical depth, tailwater depth (if below the top of the culvert) or the full cross section of the culvert barrel.

Additional design information for energy dissipators is included in FHWA HEC-14, Hydraulic Design of Energy Dissipators for Culverts and Channels.

10.0 Reset Castings - Manholes and Inlets

10.1 Reset Castings and Construction Practices

Where a manhole or inlet is to be raised using the item, Reset Castings and the existing hardware is excessively worn or in otherwise poor condition, a new frame and cover or grate shall be used.

The condition of the existing hardware and its probable performance after resetting needs to be assessed. If wear has caused the cover to be depressed more than 1/4 inch below the top of the frame, a new frame and cover or grate shall be specified.

On new pavement elevations exceeding 3 1/2 inches, castings shall be reset as follows: on multi-course resurfacing projects, the base and/or binder course shall be placed before a manhole frame is raised. This increases the accuracy in bringing the manhole to the proper grade and cross slope and leaves no more than 1 1/2 inches of casting exposed to traffic, thus permitting the roadway to be opened to traffic. If the specified cross slope of the overlay is different from that of the existing pavement, an extension ring with the necessary slope change built into the casting shall be specified.

For purposes of plan preparation, Cast Iron Extension Frames for Inlets and Extension Rings for Manholes shall be used to raise existing castings a maximum of 3 1/2 inches. When existing castings are required to be raised more than 3 1/2 inches to a maximum of 12 inches, the item Reset Castings shall be used. The item Reset Castings shall also be used to lower grades and elevations up to 12 inches. Adjustments of grades and elevations in excess of 12 inches will be considered as reconstructing inlets and manholes and the appropriate pay items shall be used.

Before Cast Iron Extension Frames or Rings are called for at a particular location, a determination shall be made by the design engineer as to whether the existing casting was previously raised using a Cast Iron Extension Frame or Ring, and what height was used. If a Cast Iron Extension Frame or Ring was previously used and the sum of the previous resetting plus the proposed resetting exceeds 3 1/2 inches, then the item Reset Castings or the appropriate reconstruction item shall be used.

10.2 Extension Rings and Frames

When structures contain existing frames or rings, these extension frames or rings shall be removed. Multiple extension frames and rings are not allowed.

The design engineer may decide to reset a particular head by either using the item, Reset Castings, or by installing an extension frame. This decision will primarily be influenced by the following factors:

A. The height to which the head is to be raised.
B. The maximum height of the casting above the roadway surface when open to traffic.
C. The prevailing traffic speed and volume.
D. The location of the casting in the traveled way or shoulder.
E. Expected interference with traffic flow.

F. The actual condition of the casting.
G. The comparative costs of resetting a casting (e.g. in concrete pavement, resetting is generally more expensive).

While some case-by-case analyses of these factors will be required, if the rise of head is between 1 1/2 inches to 3 1/2 inches, an extension unit will generally be specified. If the rise of the elevation is less than 1 1/2 inches or more than 3 1/2 inches, the casting will be reset by the conventional method.

10.3 Extension Rings - Manholes

On all resurfacing projects where the proposed overlay thickness is between 1 1/2 inches and 3 1/2 inches, an extension ring shall be used to reset heads.

When installing the extension ring, any rise above 1 1/2 inches must be paved over and reset before the surface course is placed unless the binder course is placed before opening the roadway to traffic.

The minimum thickness for a manhole extension ring is 1 1/2 inches. Since the Standard Manhole Cover is 2 inches thick, any height adjustments in the range of 1 1/2 inches and 2 1/4 inches will require a new Heavy Duty Cover (1 inch thick). Any salvageable cover in good condition can only be used in an extension ring 2 1/2 inch or more in height.

The following guidelines shall assist in determining where to use Extension Rings for Existing Manholes:

A. If the rise, R, is from 1 1/2 inches to less than 2 1/2 inches, an Extension Ring for Heavy Duty Cover (1 inch thick cover) is warranted.
B. If R is 2 1/2 inches to 3 1/2 inches, use a new Extension Ring for Standard Cover (2 inches thick cover).
C. If R is less than 1 1/2 inches or greater than 3 1/2 inches, use the item Reset Castings, to raise the manhole.

10.4 Extension Frames - Inlets

The minimum height of an inlet extension frame is 1 3/4 inches. Depending on how extensively depressed or "dished" an existing inlet may be, an extension of 2 inches, 2 1/2 inches, or 3 inches high may be required to enable the top elevation of the head to be set flush with the finished grade of a 1 1/2 inches overlay.

The following guidelines shall assist in determining where to use Extension Frames for Existing Inlets:

A. If R is 1 3/4 inches to 3 1/2 inches, inclusive, use an extension frame.
B. If R is less than 1 3/4 inches or greater than 3 1/2 inches, the manhole is to be raised using the item, Reset Castings.
C. In general, inlets use a standard 1 1/4 inches grate on all extension frames.

10.5 Ramping

Ramping around the reset heads prior to final paving shall be accomplished as follows:

A. On single course (1 1/2 inches and variable) projects, a circular ramp of hot mix shall be placed about the periphery of the manhole to extend 3 feet laterally and shall leave 1/2 inch of the extension ring exposed; this should avoid the occurrence of under-compacted, shoddy-appearing areas (due to feathering) when the surface course is placed.
B. For multi-course resurfacing projects, the base and/or binder course should be placed before the casting is reset. This increases the accuracy of raising the casting to be flush with the finished pavement and enables the work progress to be in greater conformity with the policy of not having more than 1 1/2 inches exposed for more than 48 hours.
C. For a 3 inch resurfacing where 1 1/2 inches is to be milled off, after milling, the bituminous ramp will be placed as for the single course in "A". The binder course will then be placed so that the casting will end up being set flush with the finished pavement grade.
D. For the occasional 2 inch overlays, ramps will be constructed as for the 1 1/2 inches course.
E. Do not reset the casting until the topmost (if more than one) bottom course has been placed so that not more than 1 1/2 inches will be exposed for more than 48 hours before bringing the pavement to grade.
F. The brickwork shall be set with a high early strength, non-shrink mortar developing a one-hour compressive strength of 2500 PSI at 70°F. The mortar should not contain any gypsum, iron particles or chlorides.

11.0 Stormwater Management

11.1 Introduction

As previously stated in Sections 1.0 and 2.0, stormwater management is an important consideration in the design of roadway drainage systems. Stormwater management practices, when properly selected, designed, and implemented, can be utilized to mitigate the adverse hydrologic and hydraulic impacts caused by NJDOT facilities and mitigate the loss in groundwater, thereby protecting the health of streams and wetlands, and the yield of water supply wells, and downstream areas from increased flooding, erosion, and water quality degradation. Stormwater management is required if the proposed roadway project disturbs one (1) or more acres of land or creates at least 0.25 acre of new or additional impervious surface.

This section will focus on design elements of structural stormwater management facilities common to proposed roadway projects, or retrofits to existing roadways, which typically include detention basins, infiltration basins, or a combination thereof. Detention basins may be either wet or dry ponds.

Additional guidance regarding the design of stormwater management facilities is presented in the Stormwater Best Management Practices Manual. All designs must comply with the appropriate regulatory requirements and the Stormwater Best Management Practices Manual.

A. Stormwater Quantity Requirements

As per the NJDEP Stormwater Rules at N.J.A.C. 7.8-5.4(a)3, Stormwater BMPs shall be designed to one of the following:

1. The post-construction hydrograph for the 2-year, 10-year, and 100-year storm events do not exceed, at any point in time, the pre-construction runoff hydrographs for the same storm events.

2. There shall be no increase, as compared to the pre-construction condition, in peak runoff rates of stormwater leaving the project site for the 2-year, 10-year, and 100-year storm events and that the increased volume or change in timing of stormwater runoff will not increase flood damage at or downstream of the site. This analysis shall include the analysis of impacts of exiting land uses and projected land uses assuming full development under existing zoning and land use ordinances in the drainage area.

3. The post-construction peak runoff rates for the 2-year, 10-year, and 100-year storm events are 50%, 75%, and 80%, respectively, of the pre-construction rates. The percentages apply only to the post-construction stormwater runoff that is attributed to the portion of the site on which the proposed development or project is to be constructed.

4. In tidal flood hazard areas, stormwater runoff quantity analysis shall only be applied if the increased volume of stormwater runoff could increase flood damages below the point of discharge.

B. Groundwater Recharge Requirements

As per the NJDEP Stormwater Rules at N.J.A.C. 7.8-5.4(a)2, stormwater BMPs must be designed to perform to the following:

- The stormwater BMPs maintain 100% of the average annual preconstruction groundwater recharge volume for the site; or
- The increase in stormwater runoff volume from pre-construction to post-construction for the 2-year storm is infiltrated. NJDEP has provided an Excel Spreadsheet to determine the project sites annual groundwater recharge amounts in both pre- and post-development site conditions. A full explanation of the spreadsheet and its use can be found in Chapter 6 of the New Jersey Stormwater Best Management Practices Manual. A copy of the spreadsheet can be downloaded from http://www.njstormwater.org.

11.2 Methodology

As previously stated in Sections 1.0 and 2.0, specific stormwater management requirements to control the rate and/or volume of runoff may be dictated by various regulatory agencies. Groundwater recharge is required by the Stormwater Management Rule. Peak runoff discharge rates may also be limited by capacity constraints of existing downstream drainage systems.

The tasks that typically need to be performed in the design of stormwater management facilities for stormwater quantity and groundwater recharge are summarized as follows:

A. **Detention Basin**

- calculate inflow hydrographs;
- calculate maximum allowable peak outflow rates;
- calculate stage vs. storage data for the basin;
- calculate stage vs. discharge curve for the outlet; and
- perform flood routing calculations.

B. **Infiltration Basin**

- Same as for detention basin except that the stage vs. discharge curve is based on the infiltration rate; and
- The basin must be designed so that the design runoff volume is completely infiltrated within 72 hours of the end of the storm.

C. **Detention/Infiltration Basin**

Same as detention basin with the following modifications:

- The infiltration rate is typically very small relative to the discharges from the outlet structure, and is, therefore, disregarded in the stage vs. discharge curve; and
- The basin must be designed so that the volume to be infiltrated is completely infiltrated within 72 hours of the end of the storm.

Inflow hydrographs shall be computed using either the Modified Rational Method or the SCS 24-hour storm methodology as described in Section 3.6, depending upon the contributory drainage area. The Modified Rational Method is described in detail in Appendix A-5 of the NJDOT's Soil Erosion and Sediment Control Standards.

The allowable peak outflow rates shall be determined as follows:

A. For regulated stormwater management facilities, i.e. requiring regulatory agency review, maximum allowable outflow rates shall be as dictated by said regulatory agency.

B. For nonregulated stormwater management facilities, i.e. NOT requiring regulatory agency review, the allowable outflow rate shall avoid an unreasonable increase in runoff resulting from the project. The peak outflow rate shall be determined for the roadway design storm and the storms with a recurrence interval of once in 2-, 10-, and 25-years. Downstream stability shall be evaluated for any proposed peak outflow rate that results in an unreasonable increase in the existing peak flow rate and appropriate action shall be taken to avoid unreasonable erosion or flooding resulting from the proposed construction.

Storage volume and outlet structure rating curve data are site specific and will vary for each pond; however, sufficient storage volume shall be provided and the outlet structures shall be configured so that outflow requirements as described in Section 11.2 are satisfied.

Flood routing calculations shall be based upon the Storage Indication Method (Modified Puls). As stated in Section 3.6 the use of computer software programs such as Pond-2, HEC-1, and/or TR-20 to perform these iterative routing calculations is encouraged. Any one of these procedures is acceptable.

A typical method to maintain the existing groundwater recharge is to provide a retention/extended detention basin or sand and vegetative filter strips. An analysis of the pre- and post- developed on-site groundwater recharge conditions can be determined by using the NJDEP’s New Jersey Groundwater Recharge Spreadsheet found in the New Jersey Best Management Practices Manual. For Groundwater Recharge, it is important that the permeability rate be tested at the location of the BMP. The BMP must have a minimum Permeability of 0.2 to 0.5 inches per hour and the BMP structure must drain in less than 72 hours. For more guidance on the design of Groundwater Recharge BMPs, see Chapter 6, of the New Jersey Stormwater Best Management Practices Manual. Chapter 6 also has guidance on the use of the Groundwater Recharge Spreadsheet Program. A copy of the Spreadsheet is located in Figures 11-1 and 11-2. The Spreadsheet can be downloaded from http://www.njstormwater.org.

11.3 Stormwater Management Facility Locations

The location of stormwater management facilities will depend on several factors such as location of receiving water course, location of roadway profile low points, groundwater elevations, etc.

The design engineer should first consider, and make maximum use of locations within NJDOT right-of-way, eg. at interchanges, ramp infield areas, wide medians, before locating facilities which require additional right-of-way. However, site/project specific constraints will ultimately dictate exact locations of stormwater management facilities.

11.4 Stormwater Management Facility Design Features

Detention ponds may be excavated depressions (cut) or diked (dammed) by means of an embankment. It should be noted that any embankment/pond that raises the water level more than 5 feet above the usual mean, low water height, or existing ground, when measured from the downstream toe-of-dam to the spillway crest on a permanent or temporary basis must conform to NJAC 7:20 "Dam Safety Standards", effective May 2, 1995.

Detention ponds shall incorporate the following design features:

A. Pond side slopes shall be 1 (vertical) on 3 (horizontal) or flatter to facilitate mowing.
B. A low flow channel shall be provided having a minimum slope of 0.5% and side slopes of 1 on 3 or flatter.
C. The pond bottom shall be graded to drain to the low flow channel at a minimum slope of 1.0%.
D. A ten (10) foot wide flat safety bench shall be provided 1 foot above the normal pool elevation in a wet pond.
E. All ponds shall be evaluated for fencing needs. The evaluation shall be submitted to the Bureau of Landscape Design and Scoping and Review for their review.
F. To the maximum extent practicable, outlet structures shall be designed so as to require minimal maintenance. Trash racks and safety grating shall be provided.
G. Dry detention ponds and the portion of a wet pond above the normal pool elevation shall be topsoiled and seeded. The Landscape and Urban Design Unit should be contacted for guidance regarding seeding requirements and additional landscaping features in and around proposed ponds.
H. The height and fluctuation of the groundwater table shall be taken into account when designing any wet or dry pond. Design of a dry pond below the seasonal high water table may result in periodic flooding of the pond.

In addition, an access ramp to the stormwater management facility may be provided to allow NJDOT maintenance personnel and equipment to enter the facility for maintenance/cleaning operations. Where an access ramp into stormwater management facilities for truck access to basin bottom and outlet structure for maintenance is required, the following criteria should be applied:

- Width: 13 feet wide; and
- 8% slope desirable, 12% maximum.

Refer to the NJDEP Stormwater Best Management Practices Manual for recommended outlet structure designs and more detailed design data for stormwater management facilities.

Figure 11-1

New Jersey Groundwater Recharge Spreadsheet
Version 2.0
November 2003

Annual Groundwater Recharge Analysis (based on GSR-32)

Select Township ↓	Average Annual P (in)	Climatic Factor
MIDDLESEX CO., PERTH AMBOY CITY	47.8	1.53

Project Name:	Sample Project
Description:	This is a test application
Analysis Date:	09/01/03

Pre-Developed Conditions

Land Segment	Area (acres)	TR-55 Land Cover	Soil	Annual Recharge (in)	Annual Recharge (cu.ft)
1	1.4	Open space	Woodstown	12.9	65,498
2	0.3	Gravel, dirt	Woodstown	6.9	7,536
3	3.5	Woods-grass combination	Woodstown	13.5	171,255
4	1.4	Open space	Keyport	13.4	68,146
5	0.5	Gravel, dirt	Keyport	7.5	13,657
6	3.3	Woods-grass combination	Keyport	13.9	165,963
7	0				
8	0				
9	0				
10	0				
11	0				
12	0				
13	0				
14	0				
15	0				
Total =	10.4			Total Annual Recharge (in)	Total Annual Recharge (cu-ft)
				13.0	492,054

Post-Developed Conditions

Land Segment	Area (acres)	TR-55 Land Cover	Soil	Annual Recharge (in)	Annual Recharge (cu.ft)
1	1.5	Impervious areas	Keyport	0.0	-
2	1.6	Gravel, dirt	Woodstown	6.9	40,191
3	3.65	Open space	Keyport	13.4	177,667
4	3.65	Open space	Woodstown	12.9	170,762
5	0				
6	0				
7	0				
8	0				
9	0				
10	0				
11	0				
12	0				
13	0				
14	0				
15	0				
Total =	10.4			Total Annual Recharge (in)	Total Annual Recharge (cu.ft)
				10.3	388,620

Annual Recharge Requirements Calculation ↓

% of Pre-Developed Annual Recharge to Preserve =	100%	Total Impervious Area (sq.ft)	65,340
Post-Development Annual Recharge Deficit=	*103,435*	(cubic feet)	

Recharge Efficiency Parameters Calculations (area averages)

RWC=	3.94	(in)	DRWC=	3.94	(in)
ERWC =	0.93	(in)	EDRWC=	0.93	(in)

Procedure to fill the Pre-Development and Post-Development Conditions Tables

For each land segment, first enter the area, then select TR-55 Land Cover, then select Soil. Start from the top of the table and proceed downward. Don't leave blank rows (with A=0) in between your segment entries. Rows with A=0 will not be displayed or used in calculations. For impervious areas outside of standard lots select "Impervious Areas" as the Land Cover. Soil type for impervious areas are only required if an infiltration facility will be built within these areas.

Figure 11-2

Project Name	Description	Analysis Date	BMP or LID Type
Sample Project	**This is a test application**	**09/01/03**	

Recharge BMP Input Parameters

Parameter	Symbol	Value	Unit
BMP Area	ABMP	6656.0	sq.ft
BMP Effective Depth, this is the design variable	dBMP	5.2	in
Upper level of the BMP surface (negative if above ground)	dBMPu	-5.2	in
Depth of lower surface of BMP, must be>=dBMPu	dEXC	0.0	in
Post-development Land Segment Location of BMP, Input Zero if Location is distributed or undetermined	SegBMP	0	unitless

Root Zone Water capacity Calculated Parameters

Parameter	Symbol	Value	Unit
Empty Portion of RWC under Post-D Natural Recharge	ERWC	0.93	in
ERWC Modified to consider dEXC	EDRWC	0.93	in
Empty Portion of RWC under Infilt. BMP	RERWC	0.74	in

Recharge Design Parameters

Parameter	Symbol	Value	Unit
Inches of Runoff to capture	Qdesign	0.54	in
Inches of Rainfall to capture	Pdesign	0.67	in
Recharge Provided Avg. over Imp. Area		19.0	in
Runoff Captured Avg. over imp. Area		24.8	in

BMP Calculated Size Parameters

ABMP/Aimp	Aratio	0.10	unitless
BMP Volume	VBMP	2,873	cu.ft

CALCULATION CHECK MESSAGES

Volume Balance-> OK

dBMP Check---> OK

dEXC Check---> OK

BMP Location---> Location is selected as distributed or undetermined

Parameters from Annual Recharge Worksheet

Post-D Deficit Recharge (or desired recharge volume)	Vdef	103,435	cu.ft
Post-D Impervious Area (or target Impervious Area)	Aimp	65,340	sq.ft
Root Zone Water Capacity	RWC	3.94	in
RWC Modified to consider dEXC	DRWC	3.94	in
Climatic Factor	C-factor	1.53	no units
Average Annual P	Pavg	47.8	in
Recharge Requirement over Imp. Area	dr	19.0	in

System Performance Calculated Parameters

Annual BMP Recharge Volume		103,435	cu.ft
Avg BMP Recharge Efficiency		76.7%	Represents % Infiltration Recharged
%Rainfall became Runoff		78.3%	%
%Runoff Infiltrated		66.2%	%
%Runoff Recharged		50.8%	%
%Rainfall Recharged		39.7%	%

OTHER NOTES

Pdesign is accurate only after BMP dimensions are updated to make rech volume= deficit volume. The portion of BMP infiltration prior to filling and the area occupied by BMP are ignored in these calculations. Results are sensetive to dBMP, make sure dBMP selected is small enough for BMP to empty in less than 3 days. For land Segment Location of BMP if you select "impervious areas" RWC will be minimal but not zero as determined by the soil type and a shallow root zone for this Land Cover allowing consideration of lateral flow and other losses.

How to solve for different recharge volumes: By default the spreadsheet assigns the values of total deficit recharge volume "Vdef" and total proposed impervious area "Aimp" from the "Annual Recharge" sheet to "Vdef" and "Aimp" on this page. This allows solution for a single BMP to handle the entire recharge requirement assuming the runoff from entire impervious area is available to the BMP.
To solve for a smaller BMP or a LID-IMP to recharge only part of the recharge requirement, set Vdef to your target value and Aimp to impervious area directly connected to your infiltration facility and then solve for ABMP or dBMP. To go back to the default configuration clik the "Default Vdef & Aimp" button.

11.5 Stormwater Management Facility Maintenance

The design engineer shall prepare a Stormwater Management Facility Maintenance Plan in accordance with the New Jersey Stormwater Rule. At a minimum, the maintenance plan shall include specific preventative maintenance tasks and schedules. The maintenance plan shall include at a minimum the manufacturer's recommendation on the maintenance of their facility. Maintenance plan guidelines are available in the New Jersey Stormwater Best Management Practices Manual. Additional maintenance information is also provided in the NJDEP Stormwater Management Facility Maintenance Manual, including recommended maintenance tasks and equipment, inspection procedures and schedules, ownership responsibilities, and design recommendations to minimize the overall need for maintenance while facilitating inspection and maintenance tasks.

A copy of The Stormwater Management Facility Maintenance Plan shall be submitted to the Division of Maintenance and Operations for review. If NJDEP permits are required, the Stormwater Management Facility Maintenance Plan shall be submitted, prior to the submission of the plan to the NJDEP with the permit application(s). Upon approval of the NJDEP Permit(s), a copy of the approved permit documentation shall be provided to the Division of Maintenance and Operations.

12.0 Water Quality

12.1 Introduction

Stormwater runoff from NJDOT facilities and activities can be a potential contributor to water quality degradation of receiving waterbodies. This section will focus on the design of water quality facilities to treat runoff from roadways. Refer to the NJDEP Stormwater Best Management Practices Handbook and the NJDOT Soil Erosion and Sediment Control Standards Manual for water quality measures and recommendations which can be used for other NJDOT facilities and activities.

Stormwater BMPs shall be designed to reduce the post-construction load of TSS in stormwater runoff generated from the water quality storm by 80% of the anticipated load from the developed site. Section 12.0 and the Stormwater Best Management Practices Manual provide guidance in the planning and design of these facilities.

For those waters designated in the tables in N.J.A.C. 7:9B-4.15(c) through (h) for the purposes of implementing the Antidegradation Polices in N.J.A.C. 7:9B-4, projects involving a Category One waterbody shall be designed such that a 300-foot special water resource protection area is provided on each side of the waterbody. Encroachment within this 300-foot buffer is prohibited except in instances where preexisting disturbance exists. Where preexisting disturbance exists, encroachment is allowed, provided that the 95% TSS removal standard is met and the loss of function is addressed.

12.2 Methodology

The water quality design storm peak rate and volume shall be determined in accordance with N.J.A.C. 7:13-2.8(b)2 (PDF Format - Page 23) which currently states using either of the following:

A. One year, 24-hour storm using SCS Type III rainfall distribution; or
B. 1 1/4 inch of rainfall falling uniformly in two hours.

12.3 Water Quality Treatment Facilities and Design

As indicated in Section 1.1 water quality is an important consideration in roadway drainage system design. Water quality facilities should be designed in accordance with all the regulatory requirements that apply.

Examples of water quality measures include, but are not limited to:

- Extended dry detention ponds
- Wet ponds
- Vegetated or biofilter swales
- Constructed wetlands
- Infiltration basins/trenches
- Oil/water separators
- Manufactured Water Quality Treatment Devices

Additional guidance regarding the design of water quality facilities is presented in the New Jersey Stormwater Best Management Practices Manual and the following web site: http://www.njstormwater.org.

This section focuses on design elements of those water quality measures most applicable to roadway projects, i.e. extended dry detention ponds, wet ponds, vegetated/biofilter swales and, manufactured water quality treatment devices.

Where stormwater management facilities are proposed for roadway projects, provisions for water quality treatment should be incorporated in the facility where possible.

For example, stormwater management facilities typically contain a low level outlet for water quality storm treatment. Stormwater management for the higher intensity storms (2-year, 10-year, and 100-year) is subsequently provided above the level of the water quality storm. Note: the term "extended" indicates that the detention pond is also designed for water quality treatment.

When a detention pond is used to provide water quality treatment, the following requirements must be met:

A. Beginning at the time of peak storage within the pond, no more than 90% of the total storm volume shall be released over a 24-hour period; the rate of release shall be as uniform as possible;
B. The minimum outlet diameter, width or height is 3 inches. If this minimum outlet size does not provide for the detention times required in A above, then alternative or additional techniques for the removal of total suspended solids(TSS) shall be provided; and
C. The species of native and/or non-intrusive exotic vegetation used in the pond is approved by the Landscape and Urban Design Unit and, if required, regulatory agencies.

When treatment within a pond is not feasible, the use of vegetated or biofilter swales is permissible provided that:

A. The water velocity does not exceed 2 feet per second (fps) to allow for settlement of TSS during the water quality design storm;
B. The slope of the swale shall not be less than 0.5 percent and the length of the swale shall be of sufficient length to allow for settlement of TSS, taking into consideration the velocity, depth of flow, and expected loading of TSS, a minimum length of 300 feet should be used for swales;
C. The residence time, i.e. time within the swale, should be maximized as much as possible, with five minutes used as the absolute minimum;
D. The design flow depth in mowed swales shall not exceed 3 inches for the water quality design storm. In swales with wetlands vegetation, the depth should be at least 1 ½ inches below the height of the shortest species;
E. Trapezoidal swale bottom widths should be no less than 2 feet and side slopes should be no steeper than 2 horizontal to 1 vertical;
F. Given the above constraints, biofilters should be designed using Manning's Equation. Recommended values of Manning's "n" are 0.020 for grass biofilters regularly mowed and those with herbaceous wetland plants, and 0.024 for infrequently mowed swales, unless other information is available.

G. If the longitudinal slope of the swale is less than 2 percent or the water table can reach the root zone of vegetation, water-resistant vegetation shall be used to survive potential standing water conditions;
H. Vegetation shall be used in the swale to filter out the TSS and to provide a secondary treatment by absorption of pollutants leached into the soil. Vegetation used in the swale shall be approved by the Landscape and Urban Design Unit and, if required, regulatory agencies; and
I. Vegetated swales should not be used as the only method of water quality treatment below the final discharge of the tormwater drainage system unless there is no other feasible method of providing water quality treatment within the project area.

When other water quality measures are not feasible, the use of Manufactured Water Quality Treatment Devices are permissible. Use of Low Impact Development techniques should be utilized to the maximum extent possible. For projects that are subject to the NJDEP Stormwater Management Regulations, the design engineer must complete the Low Impact Development Checklist found in the New Jersey Stormwater Best Management Practices Manual. If the use of a Manufactured Water Quality Treatment Device is necessary to meet the minimum water quality standards, the manufactured device should be designed in accordance with the following guidelines:

A. Use of Manufactured Water Quality Treatment Devices are limited to devices approved by the New Jersey Department of Environmental Protection (NJDEP). A Complete list of Certified Stormwater Technologies approved by the NJDEP can be found at http://www.njstormwater.org. Table 12-1 is a list of devices approved by the NJDEP:

Table 12-1
Approved Manufactured Water Quality Treatment Devices

Product*	Manufacturer	TSS % Removal
Stormwater Management Inc Stormfilter	Stormwater Management, Inc.	80%
Vortechnics Stormwater Treatment System	Vortechnics Inc.	50%
High Efficiency Continuous Deflective Separator Unit	CDS Technologies	50%
Stormceptor Stormwater Treatment System	Stormceptor Group of Companies	50%
Bay Saver Separator Device	Bay Saver Technologies, Inc.	50%

*The above list represents only those treatment devices currently certified by NJDEP as of May 2005, and should not be interpreted as exhaustive, nor as an endorsement of any particular manufacturer or product. The design engineer should evaluate each product for its suitability to the particular project being designed, and is encouraged to consult periodically with NJDEP to determine whether additional products or technologies have been certified since the creation of this document.

B. Arrange the Manufactured Water Quality devices in accordance with the New Jersey Stormwater Management BMP Manual's "Guidelines for Arranging BMPs in a Series". The design of the water quality device needs to ensure that it is located such that the structure can be easily maintained (i.e. the device is not located in the middle of a busy roadway.)
C. Selection of the appropriate water quality device should take the frequency of the maintenance into consideration. Maintenance of the device, once it is determined to be performing as designed, should be performed at most twice a year and at least once a year. The use of replacement filters is to be discouraged.

D. A maintenance plan shall be developed for the manufactured water quality device. The maintenance plan shall at a minimum contain specific preventative maintenance task and schedules and be in compliance with N.C.A.C. 7:8-5.8 and the Maintenance Guidelines for stormwater management measures in the New Jersey Stormwater Best Management Practices Manual.

12.4 Scour Considerations

Scour is to be evaluated for stream encroachment and outlet pipe protection of culverts and storm sewer pipes. For stream encroachment, substructure foundations need to be investigated for scour in accordance with the AASHTO LRFD NJDOT Design Manual for Bridges and Structures, Division 1, Section 46. The investigation consists of determining what the substructures are founded on; how deep the foundation is; and a decision on whether potential scour will endanger the substructure's integrity. Local scour and contraction scour need to be considered.

Scour is to be evaluated utilizing site-specific geotechnical information (e.g., soil types, d_{50}, etc.). The following data should be assessed in determining geotechnical impacts on the scour analysis:

1.) Review subsurface information that is provided in the Geotechnical Report.
2.) Evaluate historic scour related conditions and potential scour holes at the bridge site.
3.) Soil classification – Based on laboratory tests for grain size samples, classify the soil.

Scour depths and appropriate countermeasures can be determined through the use of the Hydraulic Engineering Circular No. 18 (HEC-18), "Evaluating Scour at Bridges", HEC-20, "Stream Stability at Highway Structures", and HEC-23, "Bridge Scour and Stream Instability Countermeasures".

Outlet protection for culverts and storm sewer pipes should be designed in accordance with Section 9.0, Conduit Outlet Protection.

13.0 Sample Hydrologic and Hydraulic Calculations

A sample storm sewer hydraulic computations and hydrologic pond design demonstrate the design procedure for a simple storm sewer system and pond as shown on Figure 13-1. For this sample, design a new land service highway through a meadow in Woodbine, NJ.

Obtain T_c for overland flow to inlets 1, 3 and 4 (based on the hydraulically most distant point) (See Manual Section 3.2) Obtain T_c from Figure 13-2.

Inlet #1

Ground Cover is grass
Overland flow length = 800 ft
Elevation at farthest point = 112 ft
Elevation at inlet = 98 ft
H = 14 ft
From Figure 13-2, (overland flow T_c)
T_c = 6 minutes, multiply by 2 for grass
<u>T_c = 12 minutes</u>

Inlet #3

Ground cover is grass
Overland flow length = 980 ft
Elevation at farthest point = 98 ft
Elevation at inlet = 96 ft
H= 2 ft
From Figure 13-2
T_c = 17 minutes, multiply by 2 for grass
<u>T_c = 34 minutes</u>

Inlet #4

Ground Cover is grass
Overland flow length (farthest point from channel) = 480 ft
Elevation at farthest point = 118 ft
Elevation of channel invert = 102 ft
H= 16 ft
From Figure 13-2
T_c = 3.2 minutes
Multiply by 2 for grass
T_c =6.4 mins.
T_t through channel:
L=330 ft
H=102 ft – 93 ft= 9 ft
From Figure 13-2
T_c =2.5 min.
<u>Total T_c = 6.4 mins.+ 2.5 mins. = 8.9 minutes, use 10 minute minimum T_c</u>

Figure -13-1

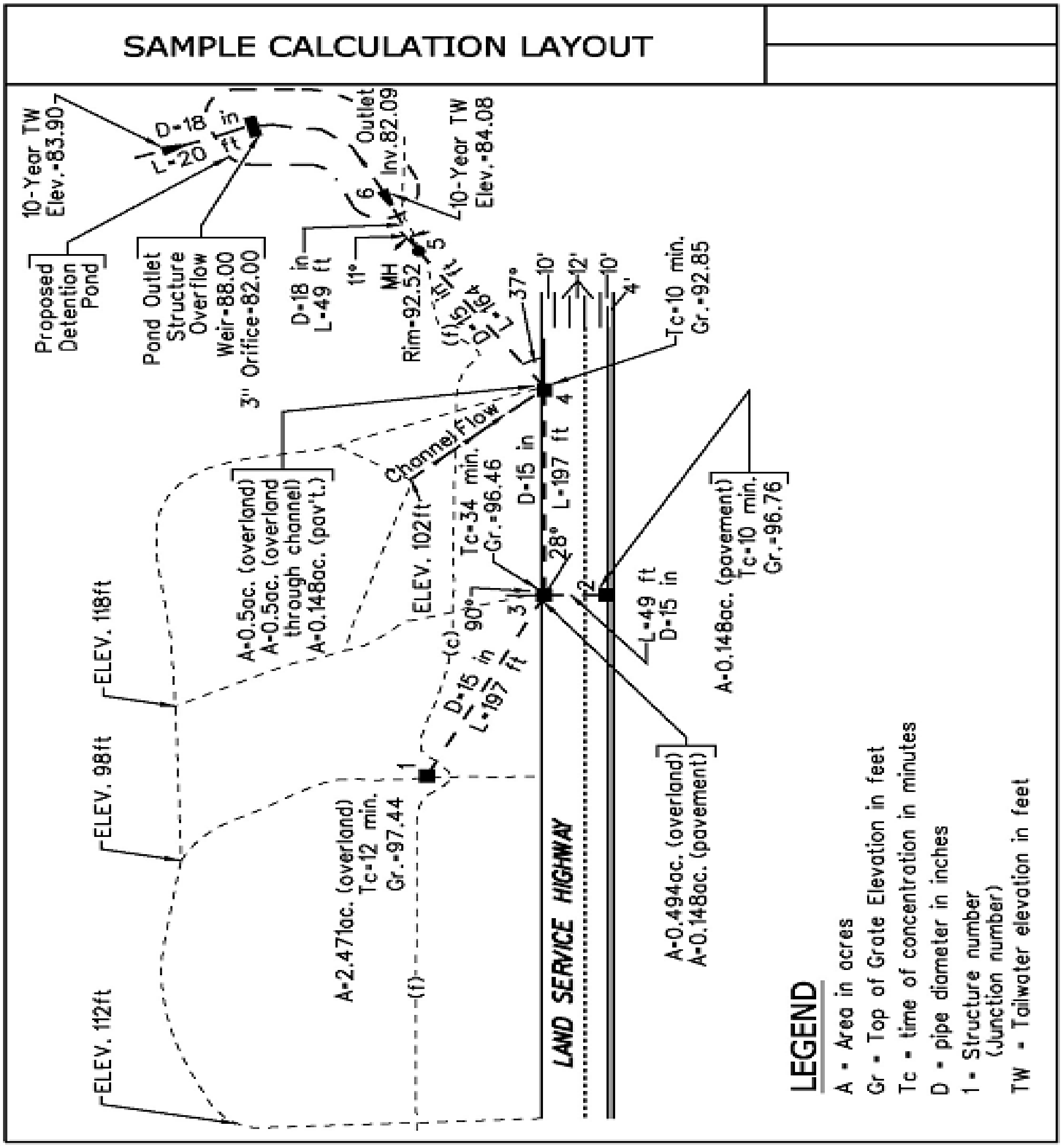
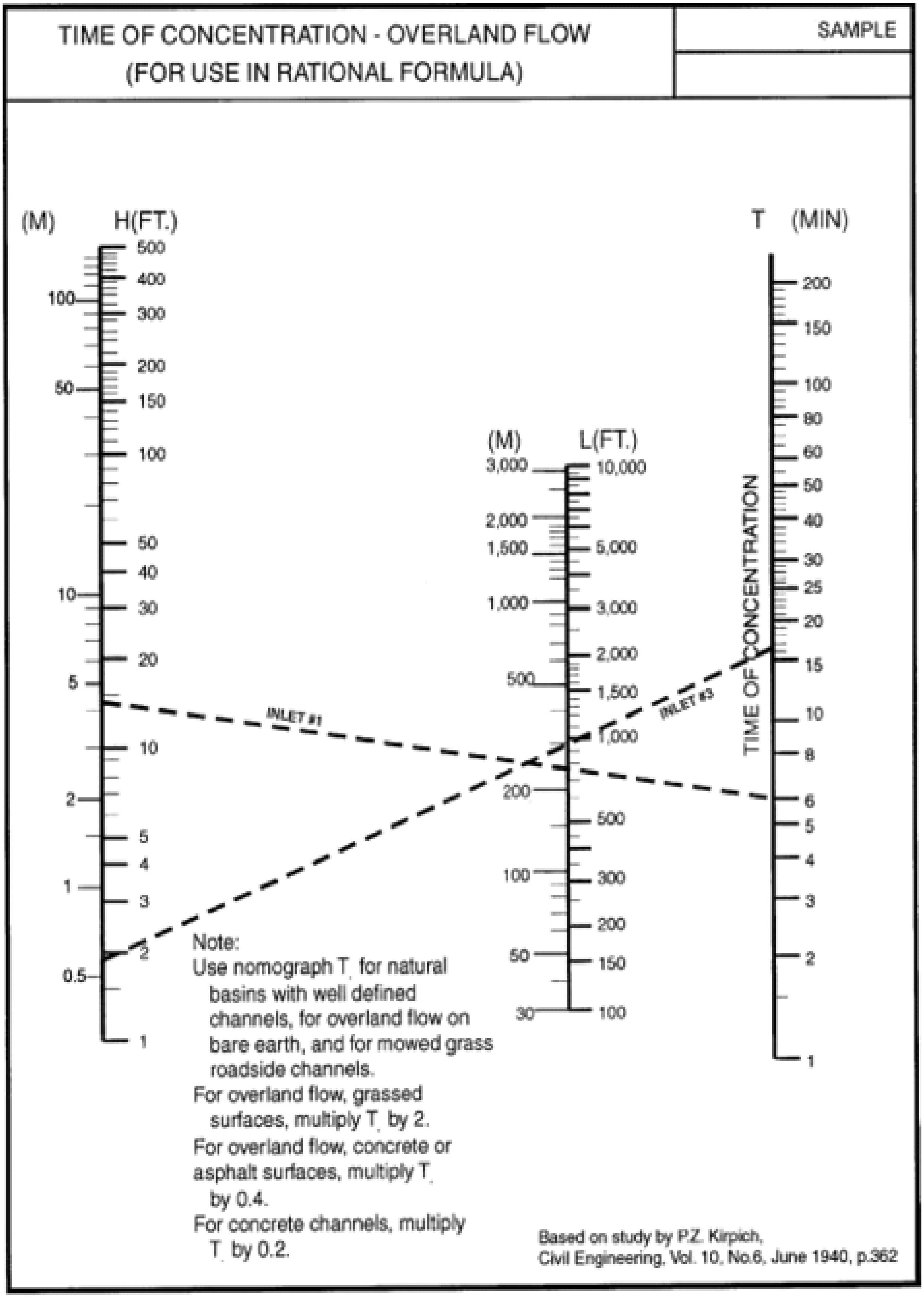

SAMPLE CALCULATION LAYOUT
Proposed Detention Pond
Pond Outlet Structure Overflow Weir=88.00 3" Orifice=82.00
10-Year TW Elev.=83.90
10-Year TW Elev.=84.08
Outlet Inv.82.09
D=18 in L=49 ft
MH Rim=92.52
Tc=10 min. Gr.=92.85
A=0.5ac. (overland) A=0.5ac. (overland through channel) A=0.148ac. (pav't.)
Channel Flow
ELEV. 102ft
Tc=34 min. Gr.=96.46
D=15 in L=197 ft
ELEV. 118ft
ELEV. 98ft
ELEV. 112ft
A=2.471ac. (overland) Tc=12 min. Gr.=97.44
D=15 in L=197 ft
L=49 ft D=15 in
A=0.148ac. (pavement) Tc=10 min. Gr.=96.76
A=0.494ac. (overland) A=0.148ac. (pavement)
LAND SERVICE HIGHWAY
LEGEND
A = Area in acres
Gr = Top of Grate Elevation in feet
Tc = time of concentration in minutes
D = pipe diameter in inches
1 = Structure number (Junction number)
TW = Tailwater elevation in feet

Figure 13-2


TIME OF CONCENTRATION - OVERLAND FLOW
(FOR USE IN RATIONAL FORMULA)
SAMPLE
(M) H(FT.)
500
400
100
300
200
50
150
100
50
40
10
30
20
5
10
2
5
4
1
3
2
0.5
1
(M) L(FT.)
3,000 10,000
2,000
1,500 5,000
1,000 3,000
2,000
500
1,500
1,000
200
500
100 300
200
50 150
30 100
T (MIN)
200
150
100
80
60
50
40
30
25
20
15
10
8
6
5
4
3
2
1
TIME OF CONCENTRATION
INLET #1
INLET #3
Note:
Use nomograph T for natural basins with well defined channels, for overland flow on bare earth, and for mowed grass roadside channels.
For overland flow, grassed surfaces, multiply T by 2.
For overland flow, concrete or asphalt surfaces, multiply T by 0.4.
For concrete channels, multiply T by 0.2.
Based on study by P.Z. Kirpich,
Civil Engineering, Vol. 10, No.6, June 1940, p.362

13.1 Sample Hydraulic Calculations

Using Rational formula, find 10-year runoff to each inlet: (See Manual Section 3.3)

Q=CIA

Refer to Table 3-2 for runoff coefficients ("C")**,** using soil group B

Using Figure 3-1, locate the project in Woodbine, NJ. The project is located in the East Region, therefore use Figure 3-4 to obtain the rain fall intensity.

Obtain rainfall intensity (I) from Figure 13-3

Inlet	T_c (min)	I (in/hr)
1	12	5.0
2	10	5.3
3	34	3.0
4	10	5.3

Inlet #1
Q_1 = (0.25)(5.0 in/hr)(2.471 acres) = **3.09 cfs**

Inlet #2
Q_2 = (0.99)(5.3 in/hr)(0.148 acre) = **0.78 cfs**

Inlet #3
$$Q_3 = \left(\frac{0.148\text{x}0.99 + 0.494\text{x}0.25}{0.642}\right)(3.0\text{ in/hr})(0.642\text{ acre}) = \mathbf{0.81\ cfs}$$

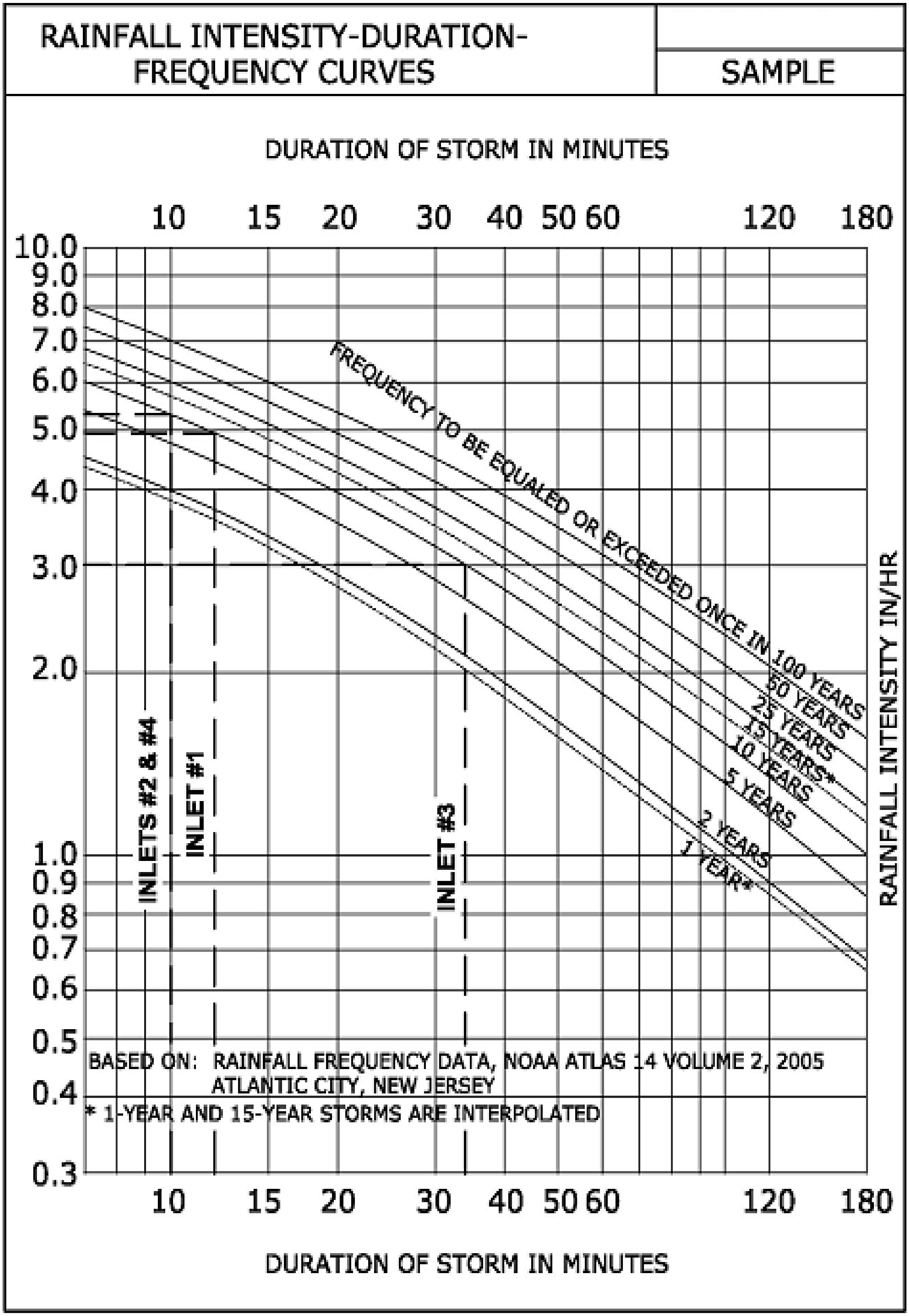
Inlet #4
$$Q_4 = \left(\frac{0.148\text{x}0.99 + 1.0\text{x}0.25}{1.148}\right)(5.3\text{ in/hr})(1.148\text{ acre}) = \mathbf{2.10\ cfs}$$

Compute gutter spread width, intercepted flow, bypass flow and efficiency for each roadway inlet: (See Manual Sections 5.5 and 5.7)

Inlet #2 (type D-1 inlet)

Q= 0.78 cfs
S_x=0.04
S= 0.03
n=0.013
T_{all}=4 ft(inside shldr. width) + 4 ft(1/3 of inside lane)=8 ft (allowable spread)

Figure 13-3



RAINFALL INTENSITY-DURATION-FREQUENCY CURVES
SAMPLE
DURATION OF STORM IN MINUTES
10
15
20
30
40
50
60
120
180
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
FREQUENCY TO BE EQUALED OR EXCEEDED ONCE IN 100 YEARS
50 YEARS
25 YEARS
15 YEARS*
10 YEARS
5 YEARS
2 YEARS
1 YEAR*
INLETS #2 & #4
INLET #1
INLET #3
RAINFALL INTENSITY IN/HR
BASED ON: RAINFALL FREQUENCY DATA, NOAA ATLAS 14 VOLUME 2, 2005
ATLANTIC CITY, NEW JERSEY
* 1-YEAR AND 15-YEAR STORMS ARE INTERPOLATED

Using a modification of the Manning equation, obtain gutter spread width:

$Q = \frac{0.56}{n} S_x^{1.67} S^{0.5} T^{2.67}$, solve for T (Section 5.4)

$T^{2.67} = \frac{0.78}{(0.56/0.013)(0.04)^{5/3}(0.03)^{1/2}}$

T=3.20 ft < T_{all} of 8 ft, OK

$y=TS_x$ (Section 5.4)

$y=3.20 \text{ ft } (0.04) = 0.128 \text{ ft}$

For the standard NJDOT bicycle safe grate, the following equation shall be used to obtain inlet interception:

$Q_i = \frac{16.88(y)^{1.54}(S)^{0.233}}{S_x^{0.276}}$ (Section 5.7)

$Q_i = \frac{16.88(0.128)^{1.54}(0.03)^{0.233}}{0.04^{0.276}} = 0.76 \text{ cfs}$

Determine bypass runoff = total runoff -intercepted runoff

Bypass flow = 0.78 – 0.76 = 0.02 cfs

(0.02 cfs would bypass to downstream inlet)

Check inlet efficiency:

$\frac{0.76 \text{ cfs}}{0.78 \text{ cfs}}$ = **0.97 > 75% , OK**

<u>Inlet #3</u> (type B inlet)

Q=0.81 cfs

s_x=0.04

S=0.03

T_{all}=10 ft

Using above equation to solve for T:

$T^{2.67} = \frac{0.81}{(0.56/0.013)(0.04)^{5/3}(0.03)^{1/2}}$

T=3.24 ft < T_{all} of 10 ft, OK

Compute inlet interception:

When T=3.24 ft, y=3.24(0.04) = 0.130 ft

$Q_i = \frac{16.88(0.130)^{1.54}(0.03)^{0.233}}{0.04^{0.276}} = 0.78 \text{ cfs}$

Bypass flow = 0.81 - 0.78 = 0.03 cfs

(0.03 cfs will bypass to inlet #4)

Check inlet efficiency:

$\frac{0.78}{0.81}$ = **0.96 > 0.75, OK**

<u>Inlet #4</u> (type B inlet)

Q=2.10 cfs + 0.03 cfs (bypass from inlet #3) = 2.13 cfs

s_x=0.04

S=0.025

T_{all}=10 ft

Using above equation to solve for T:

$$T^{2.67} = \frac{2.13}{(0.56/0.013)(0.04)^{5/3}(0.025)^{1/2}}$$

T=4.83 ft < T_{all} of 10 ft, OK

Compute inlet interception:

When T=4.83 ft, y=4.83(0.04) = 0.193 ft

$$Q_i = \frac{16.88(0.193)^{1.54}(0.025)^{0.233}}{0.04^{0.276}} = 1.38 \text{ cfs}$$

Check inlet efficiency:

$\frac{1.38}{2.13}$ = **0.65<0.75**

Since the efficiency is <75%, this inlet should be moved upstream.

When the spread width exceeds the shoulder width, the excess runoff extends into the adjacent lane, which typically has a different cross slope than the shoulder. The following example presented the computational procedure to determine the spread.

<u>Obtain spread width for a composite gutter section:</u>

Say conditions for inlet #2 are such that:

Q= 1.836 cfs

S_x=0.04ft/ft

S= 0.005 ft/ft

n=0.013

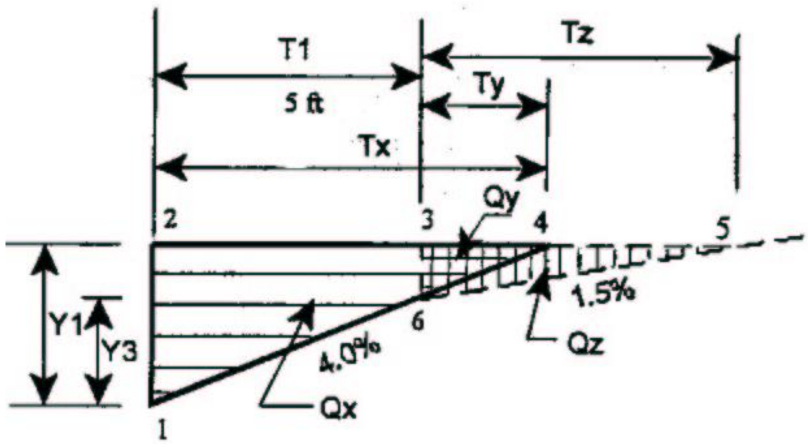
T (allowable) =5.0 ft(inside shldr. width) + 4.0 ft(1/3 of inside lane)= 9.0 ft

Using above equation:

$$T^{2.67} = \frac{1.836}{(0.56/0.013)(0.04)^{5/3}(0.005)^{1/2}}$$

T= 6.17 ft

Inside shoulder width is 5 ft, therefore, spread is beyond shoulder into adjacent through lane. Since the cross slope of the through lane differs from that of the shoulder, a composite gutter spread calculation must be performed to determine correct spread width.



Initially, a depth is assumed (y_1). Q_x,Q_y and Q_z are then calculated using the above equation. The flow contained in the composite section (Q_t) is equal to $Q_x + Q_z - Q_y$. This process is repeated until $Q_t = Q$ (actual flow in the gutter). T (actual spread width) is equal to $T_x + T_z - T_y$.

Given T_1= 5 ft, y_3=5 ft(0.04)=0.20 ft

Find Q_x (Triangle 1,2,4)

Assume y_1 = 0.25 ft , T_x = 6.25 ft

$$Q_x = \frac{0.56}{0.013}(0.04)^{5/3}(0.005)^{1/2}(6.25\text{ ft})^{2.67}$$

<u>Q_x=1.90 cfs</u>

Find Q_z (Triangle 3,5,6)

$$T_z = \frac{(y1-y3)}{0.015} = \frac{0.05}{0.015} = 3.33\text{ ft}$$

$$Q_z = \frac{0.56}{0.013}(0.015)^{5/3}(0.005)^{1/2}(3.33)^{2.67}$$

<u>Q_z = 0.07cfs</u>

Find Q_y (Triangle 3,4,6)

$$T_y = \frac{(y1-y3)}{0.04} = 1.25\text{ ft}$$

$$Q_y = \frac{0.56}{0.013}(0.04)^{5/3}(0.005)^{1/2}(1.25)^{2.67}$$

<u>Q_y=0.03cfs</u>

Q_t= 1.90 cfs+0.07 cfs - 0.03 cfs= 1.94 cfs

Q_t= Q, therefore, assumed depth is correct

Calculate T (actual spread width) (T_1 + T_z - T_y)

T=6.25 ft+3.33 ft – 1.25 ft= 8.33 ft

<u>**T=8.33 ft<9 ft, OK**</u>

Compute inlet interception:

$$Q_i = \frac{16.88(0.25)^{1.54}(0.005)^{0.233}}{0.04^{0.276}} = 1.41\text{ cfs}$$

Check inlet efficiency:

$$\frac{1.41}{1.836} = \mathbf{0.77} \geq \mathbf{0.75}, \textbf{ OK}$$

<u>**Obtain gutter spread width for inlet at low point: (See Manual Section 5.8)**</u>

Utilize same conditions at inlet #4, except s=0% (sag condition)

Q = $20.88(y)^{1.5}$ (for weir flow)

Solving for y:

$$y = \frac{Q^{0.67}}{7.58} = \frac{2.10^{0.67}}{7.58}$$

y=0.22 ft (Less than 0.75 ft, therefore use of weir equation is acceptable)

$$T = \frac{d}{S_x} \qquad (d=y)$$

When d= 0.22 ft, $T = \frac{0.22}{0.04} = 5.50$ ft

T=5.50 < T_{all} of 10 ft , OK

Compute storm drain pipe sizes for network using sample forms at end of this subsection. (See Manual Sections 6.4 and 6.5)

Backup Computations for Pipe Travel Time for Figure 6-2

Find T_c for pipe flow for partly full pipe (pipe 1-3):

(See Manual Section 3.5)

From column 12 - Q=3.11 cfs

From column 15 - Q_c = 4.95 cfs

$$\frac{3.11}{4.95} = 0.63 \text{ (63\% full)}$$

From Concrete Pipe Design Manual chart, "Relative Velocity and Flow in Circular Pipe", at 63% full, v=1.06 of full velocity.

v_{full} = 4.03 ft/s , v_{des} = 4.03 ft/s(1.06)=4.27 ft/s

$$T_t = \frac{197 \text{ ft}}{4.27 \text{ ft/s}} = 0.77 \text{ min.}, \quad T_c = 12.77 \text{ min.}$$ (12 min. to Junction 1 + 0.77 min. travel time in pipe)

Since T_c at inlet 3 from overland flow is 34.0 min. > 12.77 min., use 34.0 min.

PRELIMINARY STORM DRAIN COMPUTATION FORM [SAMPLE]	FIGURE 13-4

Computed: ____________ **Date:** ____________ **Route:** ________________________________

Section: ________________________________

Checked: ____________ **Date:** ____________ **County:** ________________________________

Station and Offset (1)		L (ft)	Drainage Area "A" (Acres)		Runoff Coef-ficient "C"	"A" x "C"		Flow Time "Tc" (min.)			Rainfall "I" in/hr	Total **Chapter 3** unoff Q=CIA ft^3/S	Dia. Pipe ft	Slope ft/ft	Capacity Full ft^3/S	Velocity ft/s		Invert Elevation	
Junction From	Junction To		Incre-ment	Total		Incre-ment	Total	Overland To Inlet	In U/S Pipe	Cum. Total in Pipe*						Flowing Full	Design Flow	U/S End	D/S End
		(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1	3	197	2.471	2.471	0.25	0.618	0.618	12.0	--	12.0	5.0	3.09	15	0.005	4.95	4.03	4.38	88.68	87.70
2	3	49	0.148	0.148	0.99	0.147	0.147	10.0	--	10.0	5.3	0.779	15	0.005	4.95	4.03	3.03	91.37	91.14
3	4	197	0.494		0.25	0.124		34.0											
			0.148		0.99	0.147			0.75	34.0	3.0								
				3.261			1.036		(Line 1-3)			3.11	15	0.02	9.90	8.06	7.39	87.60	83.66
4	5	164	1.0		0.25	0.25		10.0											
			0.148		0.99	0.147			0.39										
				4.41			1.433			34.39	3.0	4.30	15	0.006	5.42	4.42	4.97	83.56	82.58
5	6	49	--	4.41			1.433		0.37	34.76	3.0	4.30	18	0.005	8.05	4.55	4.73	82.35	82.09

- For Time of Concentration, use larger of overland flow to inlet or cumulative time in pipe.

HYDRAULIC GRADE LINE COMPUTATION FORM [SAMPLE]	**FIGURE 13-5**

Computed: ____________ **Date:** ____________ **Route:** ______________________________

Section: ______________________________

Checked: ____________ **Date:** ____________ **County:** ______________________________

Station & Offset (1)		(2)	Q (3)	V (4)	R (5)	L (6)	n (7)	h (8)	H_f (9)	H_e (10)	H_i (11)	H_s (12)	H_t (13)	TW (14)	HGL (15)	TOS (16)	CL (17)
Junction From	Junction To	Dia. ft	Flow ft^3/S	Vel. ft/s	Hydraulic radius ft	Length ft	Manning's	Vel. Head ft	Fric. Loss ft	Exit Loss ft	Entr. Loss ft	Struct. Loss* ft	Total Head Loss ft	Tail-water Elev. ft	Head-water Elev. ft	Top of Struct. Elev. ft	TOS-HGL ft
6 (outlet)	5	18	4.30	2.43	0.375	49	0.012	0.092	0.069	0.092	--	0.01	0.171	84.08	84.251	92.52	8.269
5	4	15	4.30	3.50	0.312	164	0.012	0.190	0.616	--	--	0.08	0.696	84.251	84.947	92.85	7.903
4	3	15	3.11	2.53	0.312	197	0.012	0.099	0.387	--	--	0.03	0.417	84.947	85.364	96.46	11.096
3	2	15	0.779	0.63	0.312	49	0.012	0.006	0.006	--	0	--	0.006	85.364	85.37	96.79	11.42
3	1	15	3.09	2.52	0.312	197	0.012	0.099	0.384	--	0.020	--	0.404	85.37	85.774	97.44	11.666

h = Velocity head, = $(V)^2/2g$ H_i = Entrance Loss = $K_i(V)^2/2g$ Refer to Table 6-3 for values of K_i

H_f = Friction Loss, = $\dfrac{29.1N^2L}{R^{1.33}} \times \dfrac{(V)^2}{2g}$ H_e = Exit Loss, $H_e = (V)^2/2g$

* For structural (junction) losses in inlets, manholes, see Figure 13-6.

STRUCTURAL AND BEND LOSS COMPUTATION FORM [SAMPLE]	FIGURE 13-6

Computed: ____________ **Date:** ____________ **Route:** ____________

Section: ____________

Checked: ____________ **Date:** ____________ **County:** ____________

(1)	(2)	Q (3)	v (4)	$\frac{v^2}{2g}$ (5)	(6)	(7)	K_s (8)	H_s (9)	A (10)	K_b (11)	H_b (12)	$H_s + H_b$ (13)
Junction Station & Offset	Downstream Dia. ft	Downstream Flow ft^3/S	Downstream Velocity ft/s	Velocity Head ft	Junction Type (L,N or O)	Flow Type (P or O)	Structural Loss Coeff.	Structural Loss ft	Angle deg.	Bend Factor	Bend Loss ft	Structural Loss + Bend Loss ft
6	--	--	0	0	--	--		--	--	--	--	--
5	18	4.30	2.43	0.09	N	P	0.3	0.03	11	0.15	0.01	0.04
4	15	4.30	3.50	0.19	N	P	0.3	0.06	37	0.41	0.08	0.14
3	15	3.11	2.53	0.10	L	P	1.0	0.10	28	0.33	0.03	0.13
		3.11	--	--				--	--	--	--	--
2	15	0.779	0.44	0.003	N	P	--	--	--	--	--	--
1	15	3.09	1.75	0.05	N	P	--	--	--	--	--	--

H_s = Structural Loss = $K_S \times \frac{(V)^2}{2g}$, K_s from Table 6-4

H_b = Bend Loss = $K_b \times \frac{(V)^2}{2g}$, K_b from Figure 7-1

NOTES:

1) Junction Type
   - L = with Lateral
   - N = with No Lateral
   - O = with Opposed Laterals

2) Flow Type
   - P = Pressure
   - O = Open Channel

13.2 Sample Hydrologic Calculations

For the same project, design a pond so that the post-construction peak runoff rates for the 2-year, 10-year, and 100-year storm events are 50%, 75%, and 80%, respectively, of the pre-construction rates. Due to the complexity of designing a pond, use of computer software is encouraged. In this example, software was used and the input and output is summarized below.

Determine what the pre-construction and post-construction discharges are without a detention pond.

Using TR-55 the discharges are the following:

Area Name	Area (Acre)	Curve Number	2-Year Flow (cfs)	10-Year Flow (cfs)	100-Year Flow (cfs)
Existing 1	2.471	58	0.51	2.72	8.96
Existing 2	0.148	58	0.03	0.16	0.54
Existing 3	0.642	58	0.13	.71	2.33
Existing 4	1.148	58	0.24	1.27	4.16
Total	**4.409**	**--**	**0.92**	**4.86**	**15.99**
Proposed 1	2.471	58	0.51	2.72	8.96
Proposed 2	0.148	98	0.40	0.63	1.09
Proposed 3	0.642	67	0.26	.76	1.97
Proposed 4	1.148	63	0.47	1.73	4.90
Total	**4.409**	**--**	**1.45**	**5.52**	**16.20**

With the aid of computer software, design a pond so that the post-construction peak runoff rates for the 2-year, 10-year, and 100-year storm events are 50%, 75%, and 80%, respectively, of the pre-construction rates. The pond should have design flows as follows:

	Inflow (cfs)	Design Discharge (cfs)
2-year	0.92	0.46
10-year	4.86	3.65
100-Year	15.99	12.79

Design a pond with a bottom length of 75 feet, bottom width of 40 feet and with 2:1 side slopes. The pond will be located as shown on Figure 13-1. The outlet structure will be a riser inlet box with a 3" orifice at elevation 82.0 feet and an overflow weir at elevation 88.0 feet. The outlet pipe from the outlet structure is an 18" reinforced concrete pipe. Note that different pond and outlet structure configuration may need to be tried for the pond to perform at the design discharge. Use of a pond sizing wizard may be helpful in determining a starting point. If the required pond size is too large for the proposed project, multiple smaller ponds may be used for detention. The pond, as designed, has the following discharges:

	Discharge (cfs)	% of Pre-Construction Rates	Check
2-year	0.16	17	OK
10-Year	3.60	74	OK
100-year	11.02	69	OK

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