EVALUATION OF BEST MANAGEMENT PRACTICES AND LOW IMPACT DEVELOPMENT FOR HIGHWAY RUNOFF CONTROL

User's Guide for BMP/LID Selection
(Guidelines Manual)

Prepared for
National Cooperative Highway Research Program
Transportation Research Board
National Research Council

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Oregon State University
University of Florida
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FOREWORD

This *Guidelines Manual* for National Cooperative Highway Research Program (NCHRP) Project 25-20(01) is one of three reports that present results of this project, which ran between August 2002 and May 2006. The overall objective of the project is to provide the highway engineer selection guidance toward implementation of best management practice (BMP) and low impact development (LID) facilities for control of stormwater quality in the highway environment. This *Guidelines Manual* presents a consolidated methodology for selecting and sizing stormwater BMPs. More detailed guidance including extensive scientific and engineering background material related to BMP/LID facilities is presented in the *Research Report*. In a third report, *LID Design Manual*, detailed design guidance is also provided for LID facilities.

This *Guidelines Manual* is intended for highway drainage engineers and similar civil and environmental engineering professionals to evaluate and select wet-weather controls in the highway environment. This document should suffice for decisions on the type of facility to install but will refer to any of many available documents for *design* of BMPs. Specific guidance on the design of LID facilities in the highway environment is provided in the LID Design Manual. Since most wet-weather controls consist of combinations of options, as in a treatment train, this *Guidelines Manual* will also provide information for selection and preliminary design of integrated LID/BMP facilities.

The project was completed through the collegial cooperation of four project team organizations:

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- GeoSyntec Consultants, Portland, OR and Boston
- University of Florida, Gainesville
- Low Impact Development Center, Beltsville, MD

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This project has been conducted in parallel with a similar effort sponsored by the Water Environment Research Foundation, namely WERF 02-SW-1, Critical Assessment of Stormwater Treatment and Control Selection Issues. By mutual agreement of the two agencies, NCHRP and WERF, portions of the content of the three NCHRP reports and of the WERF report are presented in duplicate, with similar or identical text. However, the emphasis of this NCHRP report is upon management of stormwater in the highway environment, with additional focus on LID design in the highway setting. The focus of the WERF project is management of stormwater in the broader urban environment and without the LID design focus.

ABSTRACT

The process of selecting highway BMPs has historically centered on choosing BMP types from a menu of options based primarily on reported performance and cost. However, the state of the practice allows for a more fundamental approach that explicitly incorporates the concept of unit operations and processes (UOPs) in a manner analogous to the conceptual design process for wastewater treatment systems. The incorporation of one or more UOPs into specific design elements of a stormwater BMP treatment system places emphasis on the selection of systems that are intended, by design, to specifically address project goals and objectives. These elements include conventional stormwater BMPs (e.g., swales, ponds, tanks, etc.) that provide primary and secondary UOP mechanisms (e.g. settling, filtration, adsorption, precipitation, etc.), as well as pre-treatment devices (e.g., hydrodynamic devices, trash racks, catch basin screens, etc.), custom hydraulic controls (e.g., flow splitters, weirs, orifices, etc.), and even tertiary treatment enhancements (e.g., soil amendments, selected vegetative species and microorganisms, mixing and aeration devices, and disinfection systems). The purpose of this guidance document is to provide a framework, or conceptual design methodology, for applying fundamental principles of UOPs to aid in the evaluation and selection of highway runoff management and treatment control systems. The steps of the conceptual design process presented herein include: 1) problem definition, 2) site characterization, 3) identification of fundamental process categories, 4) selection of BMPs, 5) practicability assessment, 6) sizing and development of conceptual design, and 7) development of performance monitoring and evaluation plan.

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CHAPTER 1 INTRODUCTION

Many forms of stormwater-related nonpoint source pollution from highways are associated with detrimental water quality characteristics of surface waters. Prevention or mitigation of the discharge of these pollutants has become a primary goal for many jurisdictions, including state departments of transportation (DOTs). As vehicular traffic increases, highways have become an even greater source of pollution, discharging oil and grease, heavy metals, nutrients, and sediment as a result of the vehicles themselves. But highways and their associated drainage systems also serve as a streamlined means of transport for other sources of pollution such as irrigation run-on, pesticides and fertilizers from landscape areas, and particulates from pavement breakdown. Paved surfaces also promote an increase in a variety of indirect water quality problems, such as higher temperature of discharge, increased flooding hazards, and erosion and bank instability due to limited infiltration and expedited transport of runoff.

In 1987, the Clean Water Act was revised in an attempt to address nonpoint source (NPS) pollution via the National Pollutant Discharge Elimination System (NPDES). Under the NPDES program, state DOTS are subject to waste discharge requirements for runoff from construction sites and municipal separate storm sewer systems (MS4s). These requirements are usually met through the implementation of stormwater best management practices (BMPs), which are devices, practices, or methods for removing, reducing, retarding, or preventing targeted stormwater runoff constituents, pollutants, and contaminants from reaching receiving waters (ASCE, 1999).

1.1 Background and Purpose of this Guidance

The performance and effectiveness of a BMP for treating and/or controlling stormwater runoff depend upon numerous variables, related not only to the design and operation of the system, but also the conditions of the site, techniques related to sampling, and constituents found in the water (Strecker et al., 2001). In order to ensure that BMP performance data may be transferable and comparable between locations and types of BMP systems, and thus assure that the overall evaluation and selection of a BMP is consistent, an evaluation methodology must be developed that will allow for both performance and practicability-based assessment (Strecker, 1994).

Today, many state DOTs integrate BMP design guidance into their hydraulic design manuals and some even have stand-alone stormwater management manuals with a specific focus on water quality. While many of these are quite good and provide great recommendations on choosing and sizing structural stormwater BMPs, nearly all of them lack a conceptual framework for addressing specific stormwater quality and quantity issues occurring at a particular site. Many of these manuals recommend a "black box" approach, which is based on choosing a BMP that has been shown to address the pollutants of concern and then apply "rules-of-thumb" sizing and design methods. While this is often an appropriate and valid approach, it does not adequately build upon the 100+ years of accumulated experience in the fields of water and wastewater engineering, which focus on environmental transport mechanisms and associated unit treatment operations and processes for particular water quality constituents. Furthermore, there has been significant advancement over the last 10 years in the understanding of low impact development (LID) techniques that can be drawn upon to enhance highway stormwater management.

The purpose of this manual is to provide guidance on selection, sizing, and design of BMPs and LID facilities as a function of mitigation goals, site-specific needs and constraints, regional and local characteristics, theoretical and empirical environmental transport mechanisms, and available BMP performance and cost information.

1.2 Document Organization and Design Methodology Flowchart

The document is organized according to the unit treatment processes-based design methodology recommended herein, such that each chapter is a step in the decision-making/design process. The flow chart shown below exemplifies this process, each step of which is discussed in detail below. After the recommended design methodology is presented, hypothetical examples of how to apply it to real-world situations are provided in Appendix B. The accompanying Final Research Report includes results and discussions of unit operations and processes (UOPs) research that relate to the topics covered in the main text of this guidance document.

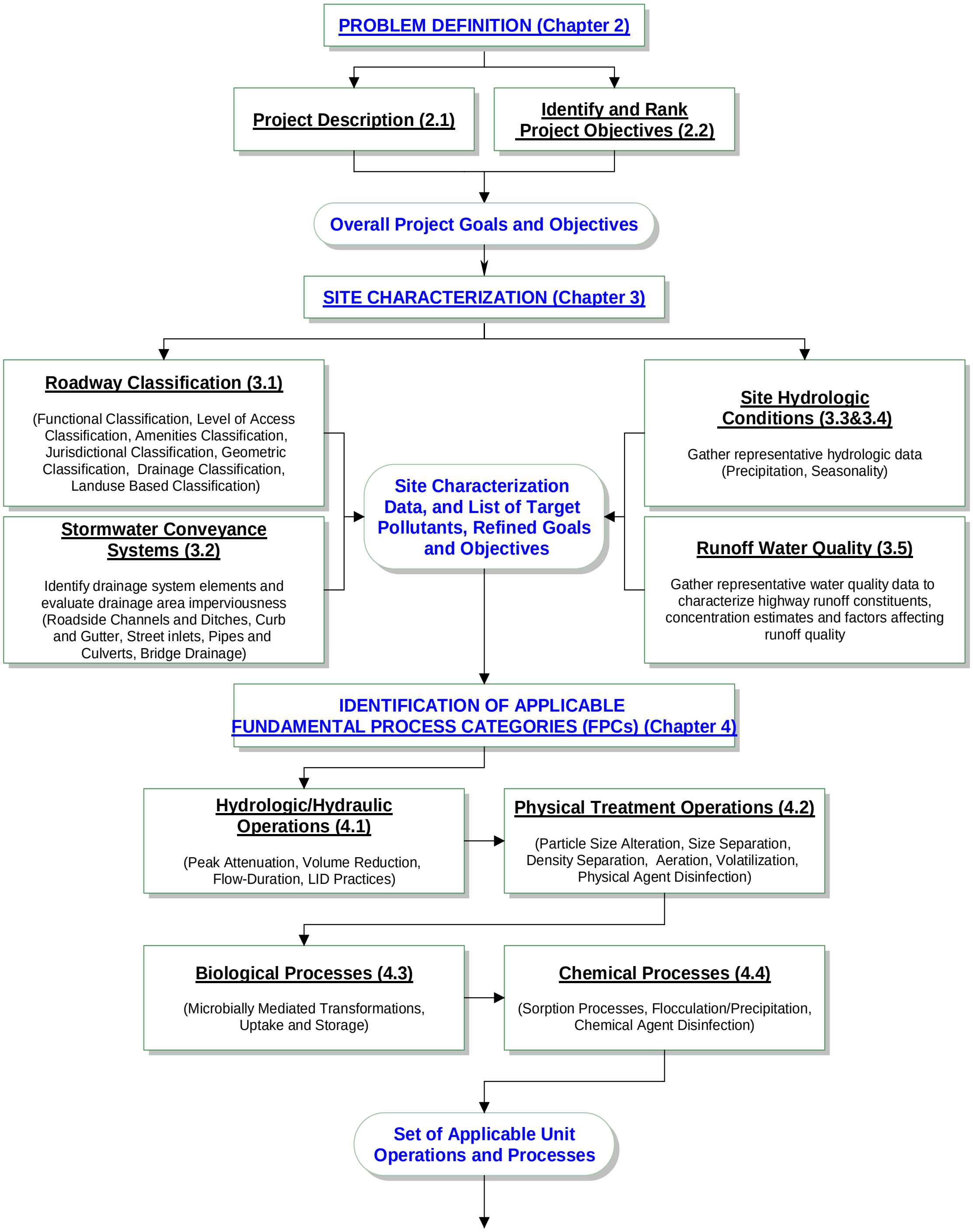

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Figure 1-1. Conceptual stormwater treatment system design methodology flow chart.

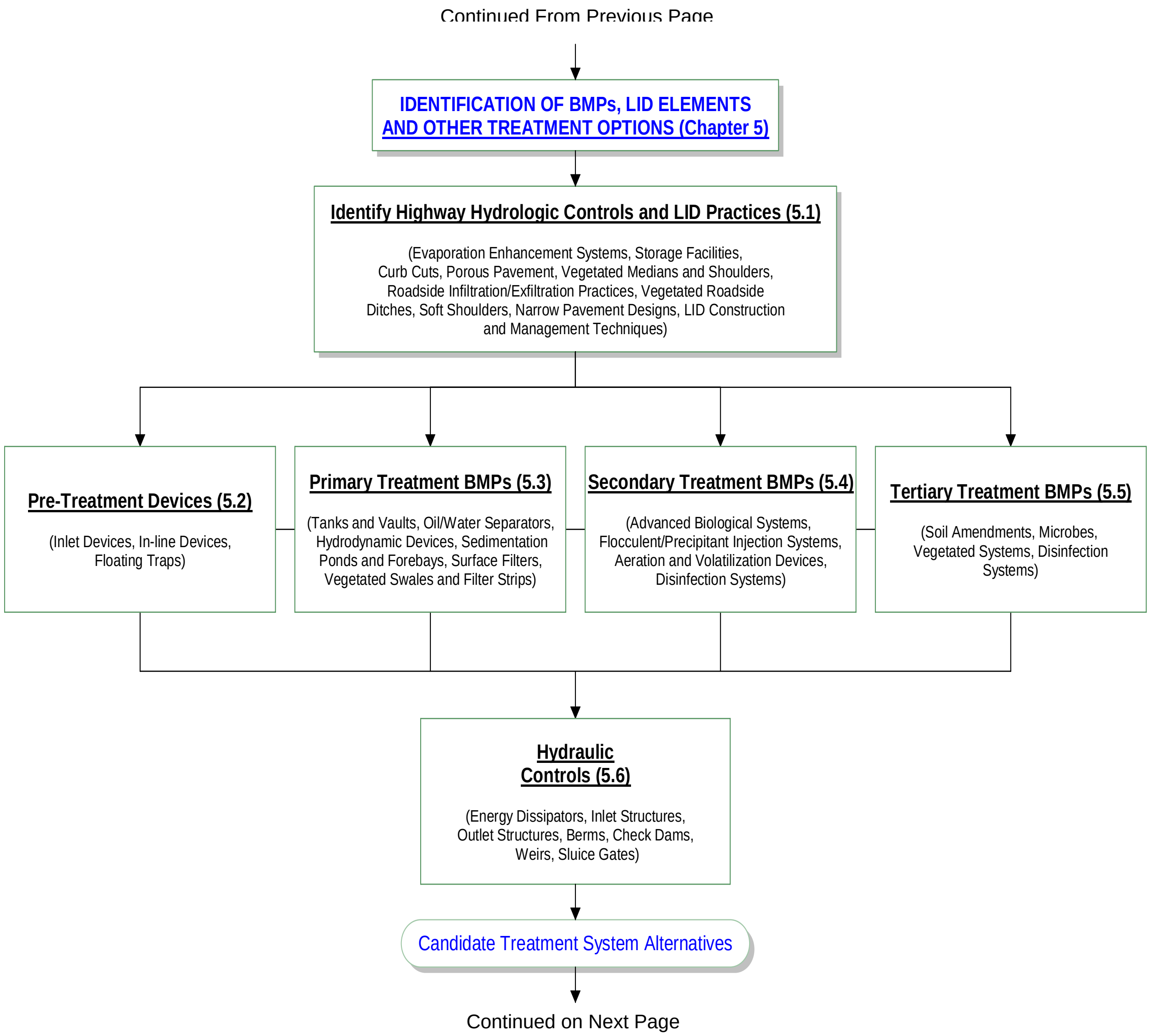

Figure 1-1. Conceptual stormwater treatment system design methodology flow chart.(cont.)

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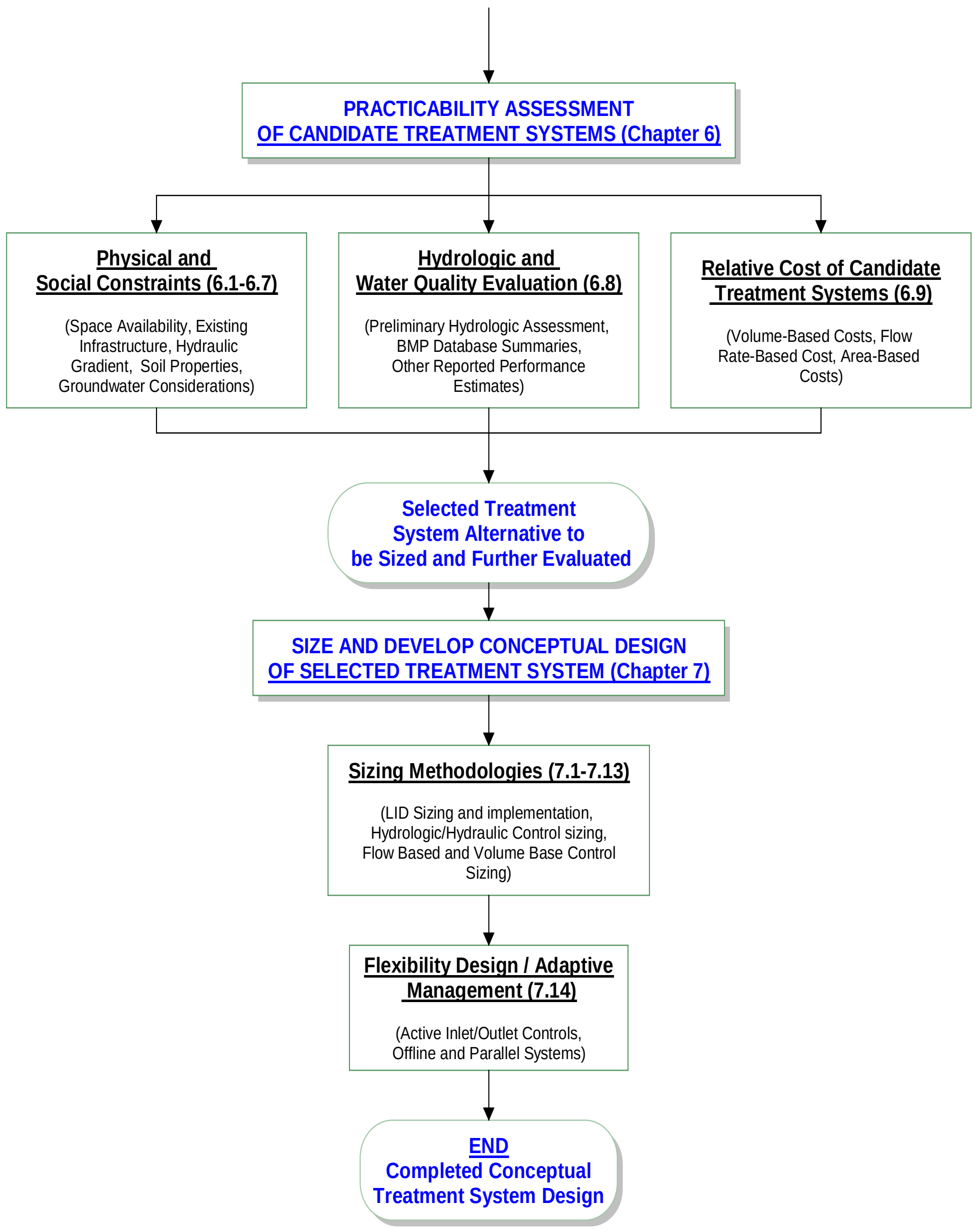

Figure 1-1. Conceptual stormwater treatment system design methodology flow chart. (cont.)

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CHAPTER 2 PROBLEM DEFINITION

The first step in any engineering project is to clearly define the problem. Without clear descriptions of the stormwater and dry-weather runoff issues that need to be addressed at a particular site including the desired results, it is difficult (if not impossible) to determine the necessary steps in selecting and designing a practicable and cost-effective highway runoff treatment system. The following paragraphs describe the primary elements of a clearly defined stormwater management project.

2.1 Description of Project

As part of the problem definition, the project should be described in detail. Sometimes a project may simply include the design of a runoff treatment system for an existing transportation corridor. For this situation, typically all of the goals and objectives of the project are directly related to runoff management. However more often than not, runoff management is only an integral part of a highway construction or renovation project, and the runoff management goals may conflict with other project goals. A clear project description will help identify where these potential conflicts may arise and may aid in coordinating planning and design activities among the various project managers and subcontractors.

Regardless of the type of project, some of the important details to include in the project description include:

- primary goals of the project (not necessarily runoff management goals),
- land availability and current and future property ownership,
- project activities including the approximate extent and timing of grading and construction, and location in terms of jurisdictional and watershed boundaries.

A clear description of the project, early in the planning phase, will help identify potential constraints and conflicts and possibly save significant time and resources during subsequent project development and implementation steps.

2.2 Identify and Rank Project Objectives

After the project has been described, the runoff management objectives, including flood control should be identified and prioritized. Table 2-1 includes a list of typical urban runoff management objectives. Frequently regulatory drivers receive top priority when establishing project objectives. Consequently, receiving water quality criteria, discharge permits, and all applicable federal, state, and local regulations should be identified early to clearly define the design problem consistent with current or future regulatory objectives. There may be several different regulations that must be met for any given project. Stormwater regulations often dictate hydrologic, hydraulic, water quality, or even design goals. The primary regulations that affect runoff management goals are summarized in Section 2.2.2.

Table 2-1. Urban runoff management objectives checklist.

Category	Typical Objectives of Urban Runoff Management Projects
Hydraulics	Improve flow characteristics upstream and/or downstream of BMP
Hydrology	For flood mitigation, improve runoff characteristics (peak shaving)
	Water balance
Water Quality	Reduce downstream pollutant loads and concentrations of pollutants
	Improve/minimize downstream temperature impact
	Achieve desired pollutant concentration in outflow
	Remove litter and debris
	Achieve loading targets
Toxicity	Reduce acute toxicity of runoff
	Reduce chronic toxicity of runoff
Regulatory	Comply with NPDES permit
	Meet local, state, or federal water quality criteria
Implementation	For non-structural BMPs, ability to function within management and oversight structure
Cost	Minimize capital, operation, and maintenance costs
Aesthetic	Improve appearance of site
Maintenance	Operate within maintenance, and repair schedule and requirements
	Ability of system to be retrofit, modified or expanded
Longevity	Long-term functionality
Resources	Improve downstream aquatic environment/erosion control
	Improve wildlife habitat
	Multiple use functionality
Safety, Risk and Liability	Function without significant risk or liability
	Ability to function with minimal environmental risk downstream
	Spill prevention
Public Perception	Provide information to clarify public understanding of runoff quality, quantity and impacts on receiving waters

Adapted from ASCE/EPA (2002).

2.2.1 Hydrologic and Hydraulic Objectives

Many state and local jurisdictions have specific hydrologic and hydraulic objectives and requirements that may have a significant affect on the size, and potentially even the selection, of stormwater treatment and control facilities. These requirements may be volumetric, flow rate-based, or a combination of both. Example volumetric requirements include treating the first 0.5 inches of runoff, capturing the 6-month, 24-hour storm, or capturing 85% of the runoff volume from an average year. Example flow rate-based requirements may include matching the 2-year, 24-hour pre-development peak discharge, treating twice the 85th percentile hourly rainfall intensity, or capturing the runoff flow rate produced from a 0.2 inch per hour rainfall intensity. While this design manual does not advocate event-based, design storm methods these requirements are presented to address the fact that they are commonly encountered by practitioners throughout the United States and may have a significant influence on the selection, sizing, and design of stormwater BMPs.

2.2.2 Water Quality Objectives

Current receiving water quality and local regulations should be identified early in a project to ensure the chosen control system meets the stipulated constraints of the site and project. In urban areas, the road drainage is often shared with the riparian land owners leading to a need to separate out the services provided by each of the owners of the local system. Available literature and data on the watershed, particularly near the site, should be reviewed to identify the existing quality of the receiving water bodies, any sensitive or endangered habitats or species, and any watershed or water body specific regulations or water quality objectives. Prior to the literature and data review, the laws and regulations typically applicable to transportation-related projects should be known. A summary of laws and regulations affecting DOT water quality management follows:

2.2.2.1 Section 303d and Total Maximum Daily Loads

Under section 303(d) of the CWA (Clean Water Act), states, territories, and authorized tribes are required to develop lists of waters that are impaired or do not support one or more of their designated beneficial uses. Waters are considered impaired if, through monitoring and assessment, they are determined not to meet established water quality standards or objectives that have been approved by the U.S. Environmental Protection Agency (EPA). The law also requires that the regulatory agency (e.g., from a state, territory, or authorized tribe) establish priority rankings for waters on the 303(d) lists and develop total maximum daily loads (TMDLs) for these waters. A TMDL specifies the maximum amount (either as a load or concentration) of a pollutant that a waterbody can receive and still meet water quality standards, and allocates pollutant loadings or concentration limits among point and nonpoint pollutant sources. By law, EPA must approve or disapprove lists and established TMDLs. If the EPA deems a submission inadequate, the EPA must establish the list or the TMDL.

State DOTs may be directly affected by these 303(d) listed water bodies and waste load allocations (WLAs) established as part of TMDLs. If a receiving water body is listed as impaired for a particular stormwater constituent, stormwater management and control efforts must focus on removing that constituent. If a TMDL has been approved and a WLA for the DOT established, the level of stormwater management and control necessary to address the load allocated to a source can be estimated. However, if a water body is listed as impaired, but a TMDL has not yet been established, the DOT may have to provide the maximum level of control possible to ensure further impairment does not occur. In either case, if receiving waters are included on a 303(d) list, the type of BMP selected and the design components chosen for that BMP should be based on the unit operations and processes known to treat the impairing pollutants.

2.2.2.2 National Pollutant Discharge Elimination System Permit Program

As authorized by the Clean Water Act Section 402, the National Pollutant Discharge Elimination System (NPDES) permit program controls water pollution by regulating point sources (i.e., discrete conveyances such as pipes or man-made ditches) that discharge pollutants into waters of the United States.

The NPDES permit program requires operators of large, medium and regulated small municipal separate storm sewer systems (MS4s) to: 1) obtain an NPDES permit, and 2) develop a stormwater management program designed to prevent pollutants from being discharged into the MS4 (or from being dumped directly into the MS4) and local waterbodies. A medium MS4 is a system that is located in an area with a population between 100,000 and 249,999. A large MS4 is a system that is located in an area with a population of 250,000 or more. Regulated small MS4s are defined as all small MS4s (MS4s not designated as medium or large) located in "urbanized areas" (UAs) as defined by the Bureau of the Census, and those small MS4s located outside of a UA that are designated by NPDES permitting authorities. An NPDES permit is also required for all construction sites that disturb greater than one acre in size.

Most states are authorized to implement the NPDES permit program, but the EPA remains the permitting authority in a few states, territories, and on most Indian land. State transportation departments are usually designated as co-permittees to municipal NPDES permits issued from the state or EPA. However in some cases a separate state-wide NPDES permit is issued directly to the state DOT (e.g., Oregon DOT).

Regardless of who is designated as the principal permittee, NPDES permits place specific stormwater management, monitoring, and reporting requirements on state DOTs, all of which may have a direct influence on BMP selection and design. The regional differences of NPDES permit requirements are highly dependent on the regional and local regulations, as well as the designated quality or sensitivity of the receiving waters as established under the 303(d) and TMDL programs discussed above. For example, many NPDES permits require performance monitoring of stormwater BMPs. In these instances, it may be desirable for the inlet and outlet structures of the BMP to be designed to accommodate measurement of flow rates and collection of water quality samples.

2.2.2.3 Section 404 Permit

Section 404 of the Clean Water Act establishes a program to regulate the discharge of dredge and fill material into waters of the United States, including wetlands. Activities in waters of the United States that are regulated under this program include fills for development, water resource projects (such as dams and levees), infrastructure development (such as highways and airports), and conversion of wetlands to uplands for farming and forestry. The EPA and the Army Corps of Engineers (Corps) jointly administer the program. In addition, the U.S. Fish and Wildlife Service, the National Marine Fisheries Service, and state resource agencies have important advisory roles.

Highway construction and stormwater management projects adjacent to or across waters of the United States may be required to obtain a Section 404 permit. This permit may have explicit requirements as to the types of activities, including stormwater treatment and discharge activities, in and around these designated waters. Surface BMPs that require the placement of berms, dams, and embankments may be subject to a Section 404 Permit.

2.2.2.4 Water Quality Criteria

CWA amendments, EPA regulations, and state water quality programs addressing point and nonpoint sources have continued to evolve over the years as increased knowledge is accumulated on the impacts of urban and highway development projects. Several sections of the CWA apply to urban runoff, both as a point and nonpoint source of pollution, as well as impacts of any activities that may result in the disturbance of natural wetlands, regulated by Section 404 of the Act. The following paragraphs describe relevance of these regulations to stormwater runoff and highway operations.

The water quality criteria developed in 1986 in accordance with the 1972 Federal Clean Water Act are designed to be protective of aquatic life-related beneficial water bodies. The USEPA national water quality criteria or National Toxics Rule (NTR) is designed to be adjusted for site-specific conditions that properly consider the aquatic chemistry of the constituents of concern.

Also, state water quality programs are required to designate uses for all state waters, establish criteria to meet those uses, and institute an antidegradation policy for waters that meet or exceed criteria for existing uses. The Act also requires that state water quality criteria must include both numeric standards for quantifiable chemical properties (e.g. California Toxics Rule – CTR) and narrative criteria or criteria based upon bio-monitoring. Some of states have adapted the NTR criteria (e.g., Oregon). State and regional water quality management plans are also required to identify priority point and nonpoint problems, consider alternative solutions, and recommend control measures. Ambient water quality standards are to be supplemented by discharge standards in the form of effluent limitations applicable to point and nonpoint sources.

2.2.2.5 National Estuary Program

The EPA administers the National Estuary Program under Section 320 of the Clean Water Act. This program focuses on all pollutant sources in geographically targeted, high priority estuarine waters. Through this program, EPA assists state regional and local governments in the development of comprehensive management plans that recommend priority corrective actions to restore estuarine water quality, fish populations, and other designated uses of the water (USEPA, 1991).

2.2.2.6 Coastal Zone Act Reauthorization Amendments

In an effort to develop a more comprehensive solution to the problem of polluted runoff in coastal areas, Congress expanded the 1972 Coastal Zone Management Act (CZMA) in 1990 to include a new Section 6217 entitled "Protecting Coastal Waters." Section 6217 of the Coastal Zone Act Reauthorization Amendments (CZARA) requires that states with approved coastal zone management programs develop Coastal Nonpoint Pollution Control Programs (coastal nonpoint programs). In keeping with the successful state-federal partnership to manage and protect coastal resources achieved by the CZMA, Section 6217 envisioned that nonpoint source programs developed under Section 319 of the CWA would be combined with existing coastal management programs. By combining the water quality expertise of state 319 agencies with the land management expertise of coastal zone agencies, Section 6217 was designed to more effectively manage nonpoint source pollution in coastal areas. To facilitate development of state coastal nonpoint programs and ensure coordination between states, administration of Section 6217 at the federal level was assigned to the National Oceanic and Atmospheric Administration (NOAA) and the EPA.

2.2.2.7 Safe Drinking Water Act

Infiltration BMPs are a means of restoring infiltration capacity, thereby reducing the storm runoff volume and reducing the runoff pollutant load by settling and filtration processes. Underground injection control (UIC) is another effective measure for the subsurface disposal of runoff from roadways, roofs and pavements. However, the regional water resources agencies must weigh the benefits of infiltration against potential negative impacts to groundwater resources and ensure that infiltration facilities are a viable long-term solution and meet the relevant regulatory criteria.

The primary objectives of the Safe Drinking Water Act of 1974 (SDWA), as amended extensively in 1984, are twofold: 1) to protect the nation's sources of drinking water, and 2) to protect public health to the maximum extent possible, using proper water treatment techniques. Sections of the SDWA address the unique concerns of underground sources of drinking water and controls for contamination of these sources. Section 1421 requires EPA to establish minimum requirements for effective UIC programs applying to five classes of wells. EPA has stated that all states are required to submit a UIC program, and that once established, all underground injections are unlawful and subject to penalties unless authorized by a permit. No injection will be authorized by permit if it results in the movement of fluid containing any contaminant into underground sources of drinking water (USDW), and such a permit will not be issued until the applicant can prove that discharge or disposal into the USDW will not affect drinking water integrity. This has prompted States to develop multi-faceted programs to protect groundwater resources and recharge areas that supply public water systems. Under these programs, wellhead protection strategies have also been developed. A provision of the SDWA requires protection of surface water discharges in areas designated as sole or principal source aquifers.

The SDWA set forth criteria for identifying critical aquifer protection areas (CAPAs), which are sole or principal source aquifers and which are determined to be vulnerable to contamination due to local hydrologic or geologic characteristics and/or potential for contamination. The regulations allow states or municipalities to designate such aquifers as “CAPAs,” thereby providing for development of an area-wide groundwater protection program. The program may identify actions in the protection area that would avoid adverse effects on water quality, and place limits on federal, state, and local government financially-assisted activities and projects that may contribute to degradation of such groundwater resources or to any loss of natural surface and subsurface infiltration and purification capabilities (SDWA).

While the program is essentially non regulatory, federal financial support for projects may be withheld if harm to the designated aquifer may occur. Mitigation measures for activities that may contaminate the aquifer (including highway runoff) are typically required to assure federal funding of the project. Any project in a sole-source aquifer area receiving federal financial assistance must be coordinated with the regional EPA office. There are some principal aquifers in the country, such as the Edwards Aquifer in Texas, where the resource is designated as the sole or principal drinking water source for the area, and if contaminated would create a significant hazard to public health. As a result, more strict regulations apply, and projects planned in the area of the aquifer are inventoried, reviewed, and approved by the general public, local authorities, state environmental agencies, and the EPA.

2.2.2.8 Endangered Species Act

Another regionally-influenced regulation that may drive the selection and design of stormwater treatment facilities is the Endangered Species Act (ESA) of 1973. The ESA, as amended, provides a means to conserve threatened and endangered species and the ecosystems upon which they depend. The ESA directs all federal agencies to use their authorities to further the purposes of the ESA by carrying out programs to conserve threatened and endangered species. These programs are necessarily based on habitat conservation and are thus very regional in nature. For example, in the Pacific Northwest (PNW), migration of anadromous fish must not be hindered, leading to prohibitions against hydraulic structures that block fish passage. Culverts designed for fish passage are commonly encountered. If a highway drains to a perennial stream in the PNW it is likely that BMPs would have to be installed prior to entry of the drainage to the stream, in order not to inhibit fish passage. This might rule out, for example, a combined or multi-purpose control installed on the stream itself and lead to a set of smaller, distributed controls. Therefore, a careful review of the endangered species of an area, including their habitat, must be conducted during the selection and design of stormwater BMPs, particularly for projects discharging to receiving waters with federally listed species or any BMP that may provide habitat for federally listed species. Constructed wetlands are a prime example of the former.

2.2.2.9 Resource Conservation and Recovery Act

The Resource Conservation and Recovery Act (RCRA) gives EPA broad authority to regulate the disposal of hazardous wastes and encourages the development of solid waste management plans and non-hazardous waste regulatory programs by States. The EPA promulgates regulations under RCRA, but like many other federal acts, the states are encouraged to develop management programs and eventually take over enforcement responsibilities. To date, many states have chosen to allow the federal programs to suffice as the state program to avoid the expense of designing and enforcing programs. The U. S. Department of Transportation has enforcement responsibilities for the transport of hazardous wastes.

RCRA provisions may be relevant under some highway construction and maintenance projects, depending on the nature of the activity, proximity to receiving waters, and characteristics of the site. RCRA, or its state or local counterpart, applies to the proper storage, use, and disposal of solid wastes (e.g., plastics, scrap metals, wood materials, rubber, plastic), petroleum or petroleum-based products (e.g., oils, greases, etc), and other chemicals used in construction (e.g., detergents, paints, solvents, etc.). Therefore, any highway construction activities that use these materials are subject to the provisions of RCRA for use and disposal. This would include vehicle and equipment maintenance and upkeep procedures at DOT-owned facilities.

2.2.2.10 National Wild and Scenic Rivers Act

This act establishes the Wild and Scenic River System, and its purpose is limited to protection of "certain selected rivers of the Nation, which, with their immediate environments, possess outstandingly remarkable qualities." It essentially provides for a mechanism to determine if a river (or river segment) can meet certain eligibility requirements for protection as a wild and/or scenic river (Corbitt, 1990) and protects designated rivers from activities that may adversely impact those values. The Department of the Interior has ultimate authority for administering the program, but states can designate rivers for inclusion in the system. The Act's framers intended for most private land's rivers to enter the Wild and Scenic River System through the state designation and management provisions (Doppelt et al., 1993). However, the Department of Agriculture administers and designates rivers in the national forests (Corbitt, 1990).

It is the intent of the Wild and Scenic Rivers Act (WSRA) to "protect the free-flowing condition to protect its water quality, and fulfill other vital national conservation purposes." In planning for the use and development of water and land resources, Federal agencies must give consideration to potential wild and scenic river areas (Corbitt, 1990). For the purposes of WSRA, water resource actions are defined as any project or action that could affect the free-flowing characteristics of the river, e.g., dredge/fill operations, placement of riprap, etc. (USEPA Region

2, 1993). Under Section 7(a) of the WSRA, federal actions on water resources are prohibited if they result in a direct adverse effect on the characteristics that result in a river's WSRA classification. The Department of the Interior has determined that actions within a quarter-mile, or within the visual field of the designated river could have a direct impact (USEPA, Region 2, 1993).

As of 1993, 32 states have conservation programs of some form where rivers or river segments, and their associated riparian environments, are protected under state Wild and Scenic Rivers legislation. As a result, many state regulations prohibit or restrict dams, protect designated rivers from channelization or diversion, or have instituted comprehensive controls for land use planning, water quality and waste-control, transportation planning and local zoning requirements. Each state maintains its own administration over designated rivers or river segments through a state or regional authority, such as the EPA Region, National Park Service, or other state environmental agency. Authority is often delegated to local jurisdictions through the establishment of riverine or river corridor commissions.

Highway construction and operations near designated river segments are subject to restrictions developed by the state. Even if such activities are temporary, any disruptions to the normal flow of the river (e.g., dams, drainage alteration), increased sediment loads (construction areas) or significant increases to pollutant loads (e.g., increased runoff volume), may be restricted by a state-enacted WSRA regulation. Through the National Environmental Policy Act (below) and/or permitting processes, the DOT should be notified if its actions are subject to restriction under the WSRA.

2.2.2.11 National Environmental Policy Act

The National Environmental Policy Act (NEPA) establishes judicially enforceable obligations that require all federal agencies to identify the environmental impacts of their planned activities. The NEPA legislation and its requirements provide the framework under which environmental impacts of all substantial federal projects are evaluated, and have been the starting point from which many other environmental regulations are applied and enforced. Any major effort that involves federal funding, oversight, or permits, such as highway operations and projects, is subject to the NEPA process to ensure environmental concerns are considered and documented in an environmental impact statement (EIS) before implementation.

2.2.3 Implementation of BMP Practices

As demonstrated in the discussion above, the implementation of highway projects, including those with and without BMPs, is subject to a wide variety of regulations that may affect the selection and design of BMPs. Unfortunately, the lengthy environmental assessment and review process, as required by NEPA, will often cause significant delays (on the order of years) in the implementation of highway projects, some of which are essential infrastructure upgrades to reduce congestion and improve environmental quality. The primary causes of these delays are twofold: 1) the overall level and inherent redundancy of environmental assessment and review requirements by federal, state, and local laws, and 2) the proposed project design features or mitigation measures are not acceptable to the regulating authorities.

The first primary cause of delay of highway project implementation is based mainly on bureaucratic inefficiencies and jurisdictional overlap that cannot be addressed at the project level. However, steps are being made to address this issue. For instance, President Bush signed Executive Order 13274 on September 18, 2002. This order is intended to enhance environmental stewardship and streamline the environmental review and development of priority transportation infrastructure projects. In accordance with the order, the U.S. DOT developed the Transportation Infrastructure Streamlining Task Force to 1) monitor and assist agencies in their efforts to expedite the review of transportation infrastructure projects and issue permits or similar actions, as necessary; 2) review projects, at least quarterly, on the list of priority projects designated by the Secretary of Transportation; and 3) identify and promote policies that can effectively streamline the process required to provide approvals for transportation infrastructure projects, in compliance with applicable law, while maintaining safety, public health, and environmental protection. States that wish to have a highway project designated a priority project must submit nominations to the Secretary of Transportation. Projects added to the priority list generally are of national or regional importance, have a high level of support among state and local elected officials, and have been or are likely to be delayed by the federal agency review and coordination process.

The other primary cause of project implementation delay is based mainly on either an insufficient assessment of the potential environmental impacts or unfamiliarity with the proposed project design features or mitigation measures by the regulatory community. Depending on how the environmental impact assessment is presented for a project, stormwater BMPs and LID facilities may be considered either project design features or mitigation measures. In either case, it must be demonstrated that the project will not cause significant water quality impact with the implementation of the proposed controls. For new and innovative stormwater treatment technologies it may be difficult to demonstrate the performance of such technologies due to the lack of third-party evaluations, which presents a significant roadblock to the evolution of stormwater BMPs, including many LID practices. Some regulatory agencies have begun testing and providing lists of acceptable treatment technologies. For example, under the Environmental Technology Verification (ETV) Program - Wet Weather Flow Technologies Area, the EPA approves innovative treatment technologies through performance verification and dissemination of information (http://www.epa.gov/etv/index.html). Some state regulatory agencies have also developed similar programs, such as the "Stormwater Best Management Practices Demonstration Tier II Protocol for Interstate Reciprocity," which has been endorsed by California, Massachusetts, New Jersey, Pennsylvania, and Virginia (Tier II, 2001), and the "Guidance for Evaluating Emerging Stormwater Treatment Technologies, Technology Assessment Protocol - Ecology (TAPE)" for the Washington Department of Ecology (WADOE, 2002). While these programs are beginning to test and approve/disapprove innovative technologies, there are still many proprietary BMPs that have yet to be verified. If a proposed BMP is not verified, the level of acceptance by the regulatory agency may be limited, even if the fundamental unit processes provided by the BMP can be theoretically demonstrated.

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CHAPTER 3 SITE CHARACTERIZATION

After the project has been described and the objectives identified, the next step in any development, redevelopment, or retrofit project is to characterize the site conditions and constraints. This step is critical for the assessment and identification of feasible solutions to the runoff management problem. Site conditions may significantly influence the treatability and manageability of urban runoff. Through careful characterization of the hydrologic, geologic, and anthropogenic factors that affect urban runoff quantity and quality, receiving water quality, and local regulations, the applicable Fundamental Process Categories (FPCs) available for runoff management practices can be identified.

Major constraints that are likely to affect the selection and design of any project alternative should also be identified at this stage. Examples include physical constraints such as steep slopes, rocky terrain, soil drainage, or groundwater conditions that could limit the types and locations of BMPs; institutional constraints such as locally approved BMPs and design criteria, or legal or social considerations such as funding availability, property owner agreements, incompatible land uses, construction access, and sensitivity to noise, dust, and traffic during construction. At this stage of the project, opportunities and constraints should be identified at a reconnaissance level. For instance, a physical constraint such as space availability may significantly influence the selection and design of stormwater treatment and control facilities. Also, soil characteristics may affect the choice and design of several BMP types, so in-situ soil testing (e.g., hydraulic conductivity) should be conducted early in the planning stage.

For the highway environment, space availability may vary significantly, but typical open space areas include roadside embankments, medians, cloverleaves, and near on-ramps and off-ramps. In urban areas, highway right-of-way may be limited due to previous widening projects and build-out of surrounding land uses, while in rural areas there are often large open spaces adjacent to and in the medians of the primary travel lanes. The existence of open space does not necessarily constitute the availability of space for a structural BMP since planning for future expansions may take precedence over stormwater control projects. Thus, the only option for highly urbanized or urbanizing areas may be subsurface devices that can be located within the storm drain system, such as underground tanks/vaults, hydrodynamic devices, media filters, and catch basin inserts. However, these types of devices generally provide a lower level of treatment and are more difficult to monitor and maintain than surface BMPs.

After identifying project opportunities and constraints, additional information is required to select potential alternative treatment trains. Sufficient qualitative and quantitative data on anticipated pollutants and concentrations in runoff and receiving water quality and sensitivity (e.g., impairments due to specific pollutants or categories) need to be collected in order to prioritize pollutant reduction goals whether through source controls or pollutant removal through treatment. Once priority pollutants have been identified, fundamental unit processes can be selected based on their effectiveness to meet the project goals. Further consideration such as practicability of the identified processes to accommodate the project constraints and sizing of the actual system comes later in the process. Information on highway runoff quality is provided in this documentation; valuable watershed-specific information can be obtained from the following sources:

- Surf Your Watershed (http://www.epa.gov/surf/)
- Science in Your Watershed (http://water.usgs.gov/wsc/)
- Know Your Watershed (http://www.ctic.purdue.edu/KYW/KYW.html)

- State 303(d) and TMDL Websites

This section describes the methodology for identifying and assessing conditions in the project area that are relevant to the selection and design of stormwater treatment systems. A large amount of data will likely be collected in this step. However, the objective should be to collect just enough data needed for preliminary selection and design of potential treatment systems. Assessing site characteristics should involve identifying opportunities and, to a certain extent, constraints that influence the selection and design of highway runoff treatment systems. Opportunities for incorporation of LID techniques that emphasize pollutant source control through infiltration; interception, evapotranspiration and reduction of directly connected impervious area should be identified when characterizing the site conditions. These distributed LID approaches should be considered before centralized BMP treatment systems. It should be kept in mind that some LID techniques (including reduction of directly connected impervious area) will require significant modification to the design of roadway features including curb cuts, porous pavement, vegetated medians, vegetated road side ditches etc. The highway engineer must be open to alternative designs that do not significantly alter the safety or structural integrity of the project. A survey of the following catchment characteristics is important for formulating design alternatives that would potentially meet the project goals.

3.1 Roadway Classification

Numerous metrics exist for classifying or categorizing roadways. Some examples of roadway classifications include: geometric classification, functional classification, level of access restriction classification, amenities classification and jurisdictional classification. A brief explanation of each of the classification schemes is presented below. The understanding of roadway classification and associated amenities such as curb and gutters, ditches, pullouts, and sidewalks is fundamental to the selection, implementation, and integration of effective treatment systems into the highway and near-roadway environment. Please note that these classifications are not mutually exclusive; most roadways belong to one class in each of the classification schemes.

3.1.1 Functional Classification

This classification is based on the principal functions of a roadway or how the road is used. This classification scheme is useful for planners since it is based on functionality. The various roadway classes belonging to this classification scheme listed in the order of increasing importance of function are presented below:

- Urban/Rural Principal Arterials
- Urban/Rural Minor Arterials
- Urban/Rural Collector Streets
- Urban/Rural Local Streets corridor

While this guidance document is applicable to all roadway types, the focus is primarily on urban and rural arterial roadways. Stormwater runoff from low-volume, municipal roadways is typically addressed by municipal stormwater management programs.

3.1.2 Level of Access Classification

This classification is closely related to the functional classification since the level of access to a roadway influences the function of the roadway. This classification is also useful to planners and traffic engineers. The roadway classes under this categorization scheme are:

- Interstate highways with fully controlled access and grade-separated interchanges
- Expressways with controlled access including at grade intersections
- Arterials with partial or no controlled access (not the focus of this project)
- Private streets (not included in this project)
- Alleys (not included in this project)
- Cul-de-sacs (not included in this project)

3.1.3 Amenities Classification

Amenities as used here refer to the additional elements that either add functionality, or improve safety, or increase the usability of a section of roadway. The list of amenities that qualify under the above description is too long to exhaust in this document, hence a brief list of the most common roadway amenities has been used in for this classification as listed below:

- Soft or hard shoulders
- Sidewalks
- Curb and gutter
- Street lights
- Roadside ditches
- Guard rails
- Medians
- Retaining walls
- Sound walls
- Pull-outs

Each of these amenities has a specific bearing on the ability to retrofit LID controls and must be incorporated into the design of any new LID drainage and treatment components.

3.1.4 Jurisdictional Classification

Roadways are sometimes referred to as city road or a county road depending on the entity that is responsible for the roadway. Jurisdictional classification is based on the body or municipality that owns and/or maintains the roadway. A brief listing of possible classes under this classification scheme is provided below:

- State roads
- DOT roads
- Federal roads
- Private roads (not included in this project)
- City roads (not included in this project)
- County roads (not included in this project)

3.1.5 Geometric Classification

Geometric refers to the physical layout of a road section. Roads are classified based on the cross section of the road and the method of construction. This classification scheme is useful for engineers and road construction work since it based on the design and construction of a roadway. The general classes of roadway cross-sections are illustrated in Figure 3-1 and include:

- At-grade sections
- Cut sections,
- Fill sections, and
- Bench sections

Both steepness of the slope and the conditions of the terrain (e.g., whether the ground is dry or swampy) are factors that determine which type of cross section should be built at any given point during road construction to permit good cross-drainage. A detailed engineering soils analysis of a proposed roadway is also a crucial part of the highway design process. The results of the soils analysis are used to develop the design details of sections (cut and/or fill) such as depth and slope. A given section may be a combination of the above categories, e.g., cut and fill section. A cut section is often used when the road goes through a ridge that has a slope of less than 35 percent. This involves cutting earth from the ground and then either moving the earth to another area where it will be used as fill or to a disposal site. Fill sections are built on ground with slopes of up to 50 to 60 percent. Where slopes are greater than 60 percent and require drainage, a fill section is used raising the ground adjacent to a streambed for example to allow water to pass through the fill at ground level. To make a fill section, earth is taken from another section of the road (or from another area altogether) and placed on top of the existing ground. A full bench section is built on slopes of 60 percent or greater where drainage under the road is not required. The term bench-section refers to the flat bottom that is produced when the ground is cut away to create the base of the road. The material that is cut is either hauled off to an area needing fill, or it is disposed of over the roadside. Material that is disposed of over the edge of the road is not intended to support traffic, but it should be stabilized to minimize erosion of the unconsolidated material.

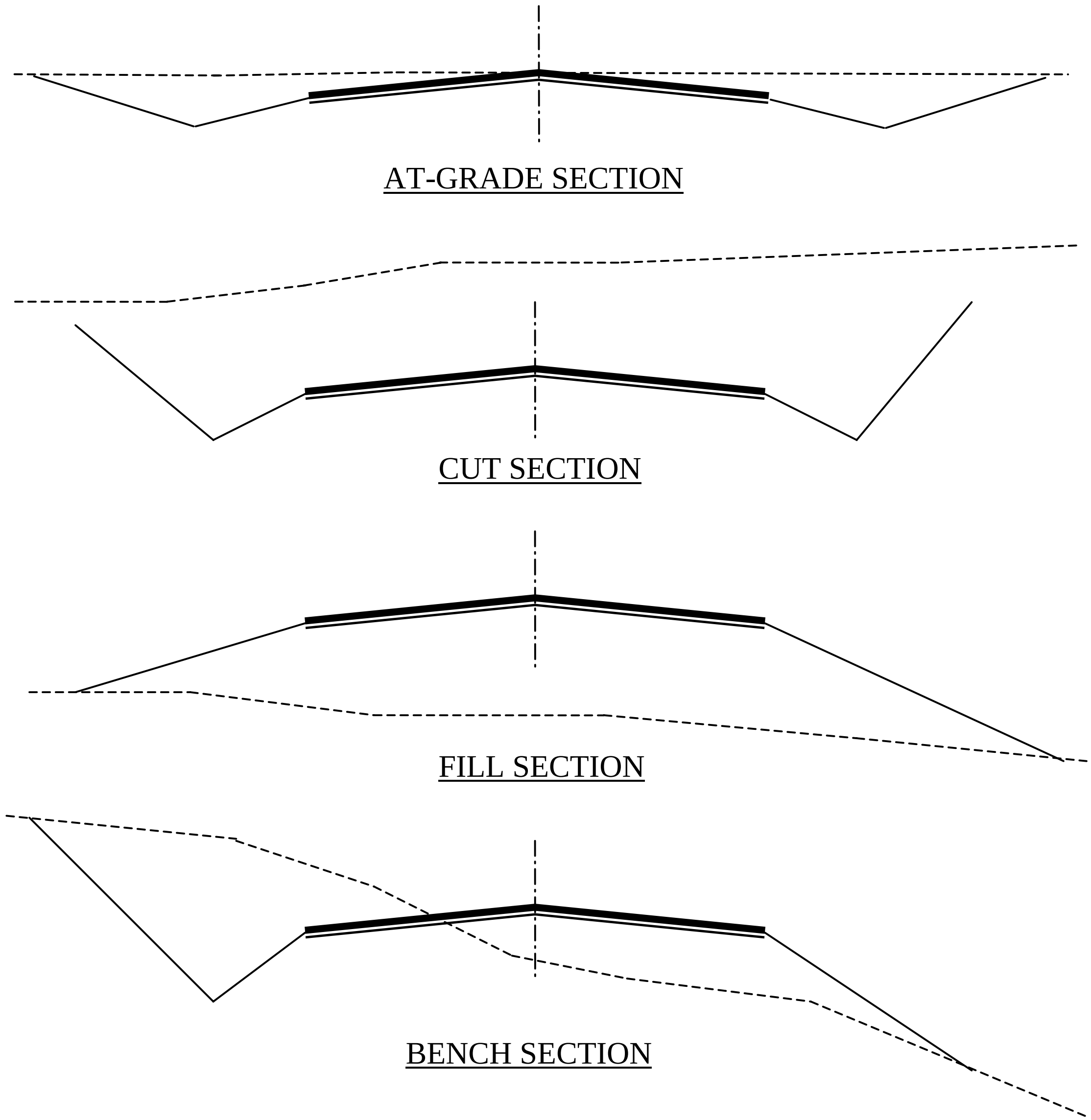

Figure 3-1. General Roadway Cross-Sections.

3.1.6 Land Use Based Classification

Two major categories of roadways based on land use classification are Rural and Urban based on a minimum population density in the service area. According to Bureau of Census, an urbanized area is defined as one having a population exceeding 50,000 people and a small urban area is designated as one having population between 5,000 and 50,000 people. Rural areas are all areas not designated Urbanized or Small Urban i.e., with a population of less than 5000 people. This classification is relevant to water quality because surrounding land uses may significantly contribute to atmospheric deposition (see Section 3.5.2).

3.1.7 Drainage Classification

Roads are also classified according to whether they have closed or open drainage. Closed drainage is associated with curb and gutter systems and associated channels and/or storm drains. In a closed drainage system, catch basins are installed approximately every 250 feet to 500 feet on both sides of the traffic lanes. Highway-surface runoff collected in these catch basins is normally piped beneath the highway to a trunkline drainage pipe beneath the median strip or road shoulder and is then conveyed into a sedimentation pool, detention basin, or other type of stormwater control systems. Figure 3-2 is a photograph of a closed drainage system in an urban setting and Figure 3-3 provides two example illustrations of cross-sections of closed drainage systems.

Figure 3-2. Photograph of a Closed Drainage System of an Urban Highway.

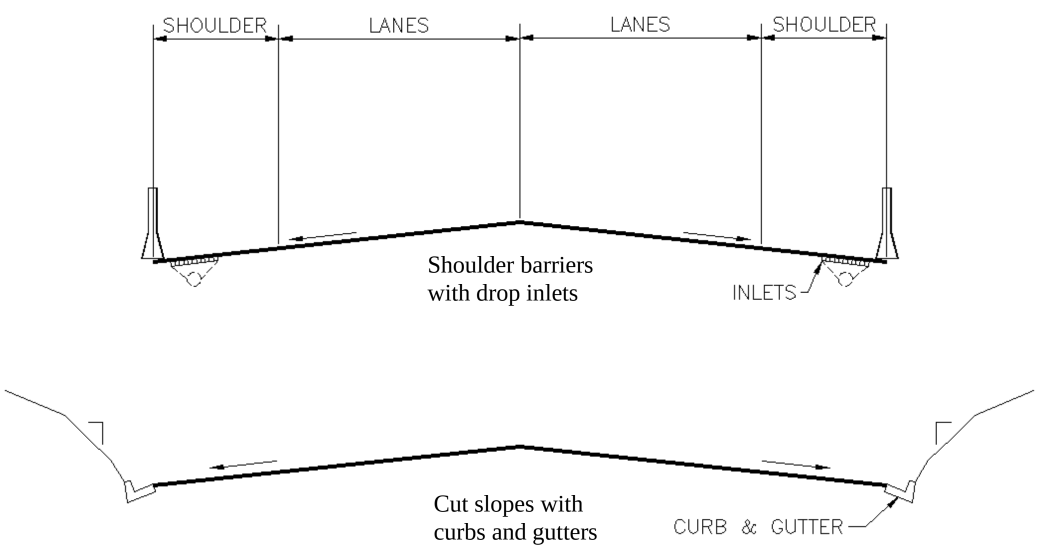

Figure 3-3. Example of Closed Drainage Systems.

Open drainage systems do not have a curb or gutter and drain to adjacent pervious areas and swales. In an open drainage system, highway runoff, whether from direct overland flow, melting of snow plowed from the highway surface, or spray caused by vehicular traffic, flows to pervious areas and is allowed to infiltrate into the soil and percolate through the unsaturated zone to the water table. Excess water flows through the open drainage system to the receiving water.

Figure 3-4. Photograph of an Open Drainage System of a Suburban Highway.

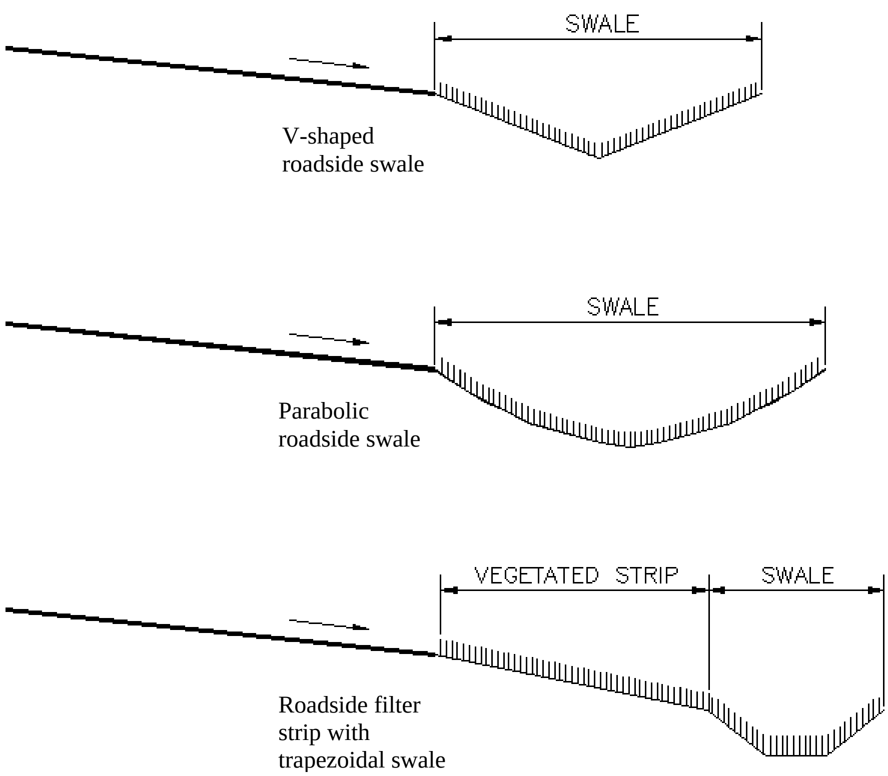

Figure 3-5. Example of Open Drainage Systems.

3.2 Stormwater Conveyance

Related to the drainage classification of roadways are the individual elements that constituent the stormwater conveyance system. The stormwater conveyance system is an important site characteristic that must be carefully examined before a stormwater management strategy can be developed. For new construction projects or projects that include the construction of a new conveyance system, there is more of an opportunity for LID designs and methods to be incorporated into the management system. Projects with closed drainage systems that can not be significantly altered are more limited to "end-of-pipe" treatment strategies using structural BMPs. The following paragraphs describe various conveyance system components and how these components relate to stormwater management.

3.2.1 Roadside Channels and Ditches

Open stormwater conveyance systems for roadways, particularly in rural areas, are often simple channels and ditches or swales (as opposed to subsurface storm drains) that collect overland flow from the paved impervious area. These simple systems often already provide water quality benefits or can be easily modified to improve water quality without compromising flood control functionality. Several important features of roadside channels and ditches may influence the design of stormwater quality retrofits including: longitudinal slopes, cross-sectional geometry, bottom and side slope material, inlet or outlet control structures, and vegetative cover.

3.2.2 Curb and Gutter

Curbs and gutters are commonplace in closed urban road drainage designs. They are installed to quickly channelize and direct stormwater away from the road surface and into the storm drain system so as to improve traffic safety and protect adjacent properties from flooding. When sidewalks are present, curbs and gutters are also installed for the convenience and protection of pedestrians. From an LID perspective, concentration of stormwater exacerbates downstream quality and quantity issues and therefore should be minimized whenever possible. Existing curb inlets can be retrofitted to provide some treatment at these locations; suitable devices are typically most effective for gross particulates and hydrocarbons.

3.2.3 Street Inlets

After stormwater is channelized in curbs and gutters it is usually directed into the storm drain system via street inlets. The type and size of the inlets are important features to consider in stormwater management, as they may affect the bulk pollutant loading rate as well as the type of treatment device (usually proprietary) that can be installed at the inlet and/or in the catch basin if one exists. The potential for inlet by-pass caused by variations in road surface micro-topography and litter build-up is also a function of the type and size of the catch basin inlet. Primary inlet types include curb inlets, drop inlets, and combination inlets. Inlet sizing and spacing are based on hydraulic efficiency with a constraint on the maximum allowable ponding of water on the traffic lanes. BMPs must not compromise this hydraulic objective.

3.2.4 Pipes and Culverts

Subsurface storm drainage systems primarily consist of a series of connected pipes and culverts. The type, size, and slope of storm drain pipes influence flow velocities within and out of the system. The specification of these features is usually founded on the goal of minimizing sedimentation within the pipes to reduce the potential for clogging and subsequent surface flooding. Upper limits on velocity may be included to reduce scour or downstream receiving water impacts. There are many opportunities to enhance water quality using public works-types of devices within the subsurface storm drain system. For example, subsurface detention in tanks, vaults, sedimentation manholes, and detention pipes (oversized pipes with mild slopes) allows sedimentation to occur upstream of surface discharge locations, reducing the sediment burden in receiving streams or regional treatment systems. These alternative, quality-based conveyance designs would require more frequent maintenance than traditional designs and subsurface removal of sediments is a relatively expensive option. However, the distribution of BMPs throughout the drainage system can decrease the required size of the system.

3.2.5 Bridge Drainage

Stormwater runoff on bridges is typically managed using scupper drains. Scupper drains allow direct discharge of runoff into surface waters below the bridge deck without treatment. For non-sensitive water bodies, this method of runoff management is probably acceptable. Bridges that are sloped enough to let water run off the ends may not need scupper drains. Alternative bridge drainage designs may include simple below-deck storm drains to convey the runoff to the ends of the bridge for treatment.

3.2.6 Drainage Area and Imperviousness Estimation

An accurate measure of the drainage area to a candidate BMP location is necessary to estimate stormwater runoff volumes and pollutant loads for the purpose of selecting and sizing BMPs. Estimating the drainage area of small, mildly sloping catchments such as a highway section is often a difficult and time-consuming task. The size of the drainage can be estimated by conducting direct field surveys with conventional survey instruments or by using the topographic maps and as-builts together with field checks for artificial barriers such as terraces and ponds. USGS topographic maps are available for all areas of a state, but are generally of too low resolution to determine flow direction on road surfaces. Also, many municipal and county entities as well as some developers have developed topographic maps of their own.

Imperviousness and runoff coefficients can be estimated using high resolution and paired rainfall and runoff data. However, typically imperviousness and/or runoff coefficients are assumed according to land use type. Land use-based imperviousness and runoff coefficients, as well as equations that relate runoff coefficients to imperviousness, are available in most introductory hydrology texts and hydrologic design manuals (e.g. Bedient and Huber, 2002; Chow et al., 1988).

Some common equations relating runoff coefficients to percent imperviousness include:

$$C = 0.05 + 0.9 \cdot I \quad \text{(Schueler, 1987)} \qquad [3\text{-}1]$$

$$C = 0.1 + 0.7 \cdot I \quad \text{(FHWA, 1990)} \qquad [3\text{-}2]$$

$$C = 0.04 + 0.774 \cdot I - 0.78 \cdot I^2 + 0.858 \cdot I^3 \quad \text{(WEF and ASCE, 1998)} \qquad [3\text{-}3]$$

C is the runoff coefficient and *I* is the total imperviousness expressed as a fraction. Note that all of these equations assume that even if the watershed is 100% impervious not all of the rainfall will result in runoff, which indirectly accounts for losses due to depression storage and evapotranspiration.

Rather than estimating the total impervious area of a watershed it is often more desirable to estimate the directly connected impervious area (DCIA). DCIA causes runoff to occur from virtually all precipitation events (Lee and Heaney, 2003). Directly connected impervious areas (DCIA) are the impervious areas such as roofs and pavement that drain directly to the street drainage system in an urban area. The impervious areas of urban transportation corridors are typically directly connected to the storm drain system. DCIA is a better measure for predicting runoff volumes because it considers the possibility of impervious runoff to infiltrate into pervious areas before discharging to receiving streams. Minimization of DCIA in order to reduce runoff volumes is the primary goal of low impact development (LID) practices. Some LID practices for the highway environment include using permeable pavements on road shoulders and in parking lots (e.g., rest areas), minimizing curb lengths or inserting curb cuts, and directing runoff to roadside and median strips and swales. These practices can usually be easily incorporated into rural highway design, but often present a challenge in the highly urbanized transportation corridors.

3.3 Precipitation

The primary factors influencing the rate and amount of runoff from a particular area are the frequency, intensity, and duration of rainfall. As both the volume and velocity of runoff increase, the capacity of runoff to detach and transport soil particles and associated pollutants also increases. Thus, from a stormwater treatment and control perspective, precipitation patterns are probably the most fundamental and important characteristics of a drainage area. While these characteristics are largely a function of other characteristics, such as climate, season, geography, etc., which are discussed later in this section, the focus of this subsection is on the acquisition and analysis of rainfall/snowfall monitoring data. This manual does not attempt to provide thorough guidance on precipitation monitoring, as this information will be incorporated by reference to monitoring guidance manuals; the attempt here is to provide a discussion of the characteristics and importance of good-quality rainfall data and how they can be analyzed to aid in the selection, sizing, and design of urban runoff controls.

3.3.1 Sources of Data

Good precipitation data, most often rainfall but sometimes snow as well in cold climates, are at the heart of any hydrologic analysis. Careful investigation of the sources and quality of such data is one of the early elements of a successful stormwater management and treatment system design project.

Precipitation data may be characterized in several ways, starting with the time series of data themselves. Precipitation as rainfall or as water equivalents where snow has fallen is routinely measured at thousands of locations across the United States. The National Weather Service (NWS) records daily weather data (either directly or more likely, through cooperators) at 26,428 sites, hourly precipitation data at 6,892 of those sites, and 15-min precipitation at a smaller subset of these sites. The central federal repository for weather data collected in this country is the National Climatic Data Center (NCDC), operated by the National Oceanic and Atmospheric Administration (NOAA) in Asheville, NC. All weather data recorded at the thousands of sites just mentioned are eventually transferred to the NCDC, with a lag time of a few to several months. The data are stored electronically, and most data (e.g., daily, monthly) may be downloaded off the Internet at no charge (www.ncdc.noaa.gov/oa/ncdc.html). Hourly and 15-minute precipitation data sometimes require a fee, although NCDC policies do change.

There are many other sources of climatic data in general and precipitation data in particular. Private companies, such as Earth Info and HydroSphere (both in Boulder, Colorado), offer climatic (and hydrologic) data from the United States and Canada for sale on CD-ROMs.

State climatological offices often provide good access to some data, although not typically to hourly or 15-min precipitation data. However, these state offices often process the data in real time, so daily records are available instantly or on the following day. The offices may be found on the Internet; for example, for Oregon at http://www.ocs.orst.edu/, which has links to other state and federal data sources. These offices may also be able to offer advice on quality control (e.g., the reliability of data from specific gages), which is not obtainable from the NCDC.

Additional sources of climatic and precipitation data include:

- local water utilities may have a rain gage at their treatment plant;
- local departments of public works and stormwater management agencies may operate rain gage networks for large cities (e.g., 23 gages for Portland, OR);
- other federal agencies and entities, such as the U.S. Geological Survey (USGS), Corps of Engineers, Bureau of Reclamation, U.S. Department of Agriculture (USDA) agencies, EPA, etc;
- state agencies, such as state environmental, agricultural, and natural resources agencies and departments;
- universities and research groups;
- television and radio stations; and
- private weather stations, some of which participate in Web-based displays (search for "personal weather stations").

Data may also be collected directly by the stormwater management professional. This has the advantage of providing site-specific data in the catchment of interest (preferably with more than one gage) and has the disadvantage of not providing a long-term record. Still, site-specific data are essential for model calibration, whereas regional data from a NWS site (often a local airport) may often be used for continuous simulation and design. Several documents related to precipitation measurement are referenced by Smith (1993) as well as in most hydrology texts.

Radar-based precipitation estimates provide a useful adjunct to point measurements (surface measurements at one gage or "point"). "Next generation radar," or NEXRAD, data from the NWS may be processed commercially to provide high resolution spatial and temporal rainfall data. Such data may be relatively costly (e.g., a few thousand dollars per extended storm event), but are invaluable for refined calibration of sophisticated models (Bedient et al., 2000).

Precipitation also includes snow. However, snow fall is a slow process that typically results in little or no runoff during an event; therefore the processes of snow accumulation and melt are typically of much greater concern. The physics of snowmelt are discussed in most hydrology texts and are simulated in some models. Accumulated snow is measured daily at NWS weather stations. On hourly or 15-min precipitation records, it must be inferred on the basis of air temperature (typically snow when the surface air temperature is less than about 1 ^{o}C or 34 ^{o}F).

Precipitation is influenced by broad geographic and climatic patterns on the earth, as is well known. A conscientious attempt to gather the most site-specific data as possible will frequently obviate the need for more complex climatological analysis.

When weather predictions are needed, reliance may be placed on NWS forecasts readily available on the internet and on any number of media (television, newspaper, etc.) web sites. If predictions need to be more precise, such as for a monitoring exercise, professional meteorologists may be hired for this purpose.

3.3.2 Data Analysis

The fundamental use of rainfall (and snowmelt) data is in the form of a hyetograph, a plot or table of rainfall intensity versus time. Such data may be used in standard hydrological techniques (such as a unit hydrograph) or used as input to models. The previous section emphasized sources of information for extraction of historic ("real") rainfall data. Design storms may also be constructed artificially by the following simple process:

1. Select a design frequency and event duration.
2. Obtain a depth from intensity-frequency-duration curves, discussed below.
3. Distribute the depth in time to create a hyetograph according to a prescribed rainfall distribution. In the United States, dimensionless 24-hr hyetographs developed by the Natural Resources Conservation Service (NRCS) are often used, such as the Type II distribution in the Southeast and the Type I-A in the Pacific Northwest (e.g., Bedient and Huber, 2002; King County, 1998).

While being an easy procedure to follow, synthetic design storms have the disadvantage of not corresponding to historic, measured storms, and have a very high intensity near the center of the 24-hr period. Also, they are burdened by the (usual, artificial) 24-hr duration. Nonetheless, because of their standardized nature and ease of construction, they are very often encountered in stormwater drainage design. However, they are usually too "extreme" for stormwater quality design in the sense of designing for too high a magnitude and too long a duration for most water quality treatment systems.

Rainfall data may also be processed in other ways. A common form of processed data is as intensity-frequency-duration (IDF) curves, an example of which is shown in Figure 3-6 for the Willamette Valley of Oregon (Oregon Department of Transportation, 1990). While fundamental to application of the Rational Method for estimating peak flows (Bedient and Huber, 2002), IDF curves are less useful for management of stormwater quality, since treatment systems are usually designed for more frequent events than, say, the 2-yr return period shown on Figure 3-6. Furthermore, depths equal to the product of intensity and duration taken from IDF curves are fundamentally a function of the selected duration, the choice of which is not obvious for other than the Rational Method (for which duration equals time of concentration). Finally, IDF curves are sometimes mistakenly thought to represent the time history of actual storms; that is, they are taken to represent a hyetograph, which is fundamentally untrue. For all these reasons, IDF curves, although very common, are not of significant use for stormwater quality analysis.

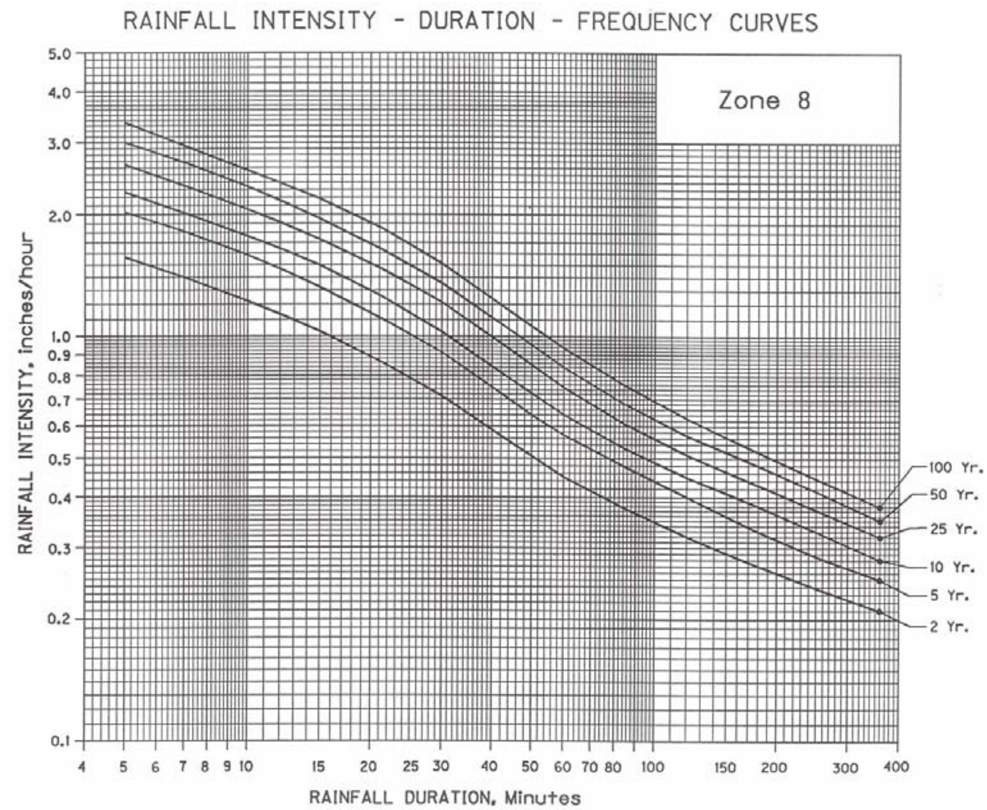

Figure 3-6. Example IDF Curve for the Willamette Valley in Oregon.

In addition to IDF curves, published precipitation summary information may also be available in the form of depth-frequency relationships, for example, magnitude versus return period, or magnitude versus percent time not exceeded. Chapter 7 provides references to design manuals some of which use this type of analysis.

3.4 Seasonality

In addition to the fluctuation of the groundwater table, there may also be significant seasonal variation in the hydrologic characteristics of highway runoff that should be considered during the planning phases of runoff control and treatment strategies. Seasonal changes in temperature, as well as variations in rainfall, help to define the high erosion risk period for the year. It is generally recognized that cold climates can influence the quality of the runoff and also the performance of the treatment BMPs. When precipitation falls as snow, pollutant transport will generally not take place until the spring. If the surface of the ground is frozen, its infiltrative capacity is reduced and runoff rates from snowmelt may be elevated. The physical and chemical variation in runoff quality has a direct influence on the treatability of constituents. Cold temperatures may cause freezing of the piping system that conveys runoff to the inlet, generate high runoff volumes during snowmelt and rain-on snow events, reduce biological activity that treats pollutants and reduce sediment settling velocities affecting removal of particulates and particulate-bound pollutants. Due to build-up of pollutants in snow pack over an entire season, high pollutant loads are likely in runoff during spring snow melt. These treatment challenges may be further complicated by seasonal regulations aimed at addressing the seasonal variation of receiving water sensitivity and recreational contact use.

The presence or absence of the "first flush" of stormwater pollution is an area of scientific debate that is briefly discussed in Section 3.5, Runoff Quality Characterization. A less disputed and related phenomenon, is seasonal flushing caused by the dry weather build-up of pollutants on impervious surfaces and the subsequent flushing of those pollutants during the first stormwater runoff event of the wet season. As opposed to "first flush" evaluations where the within-storm variance of pollutant concentrations is analyzed, seasonal flushing evaluations analyze the among-storm variance of EMCs (event mean concentrations) to assess whether initial storms of the season have substantially higher pollutant concentrations than storms occurring later in the season.

3.5 Runoff Quality Characterization

The expected quality of runoff from the catchment area should be estimated to help determine the types of BMPs and BMP systems necessary to treat the runoff. Site specific data are obviously the most appropriate, but are rarely available. More often than not the characteristics of runoff from a site must be estimated using existing data. Since there are a number of factors that may affect runoff quality, it is important to understand what site-specific conditions and regional hydrology will influence pollutant types, loadings, and concentrations. For instance, influent chemical characteristics such as pH, alkalinity, and hardness and hydrologic characteristics such as pavement residence time (both initial and average) and first flush phenomena (both seasonal and storm event) are extremely important to consider when identifying constituent speciation and phasing (dissolved or suspended) (Sansalone and Buchberger, 1997; Glenn et al., 2002). Stormwater treatability is often defined by the settling velocity (or particle size, specific gravity) distribution of runoff constituents. Larger, heavier particles are obviously easier to remove. Unfortunately, treatability data are relatively unusual in monitoring programs, but numerous generalizations are possible, as described in the following paragraphs. This section will guide the user through the process of estimating runoff quality using available monitoring data and discuss some of the factors that affect runoff quality from roadways.

3.5.1 Highway Runoff Constituents and Concentration Estimates

During the 1970s and 1980s, the Federal Highway Administration (FHWA) funded several studies pertaining to highway runoff quality characterization (Gupta, 1981; Kobriger, 1984; Kramme, 1985; Dorman et al., 1987; Driscoll et al., 1990). These studies provided the foundation for the current understanding of highway runoff quality and its impacts to receiving waters. Since the 1980s, literally hundreds of highway runoff characterization and control studies have been conducted by numerous state and federal agencies that have further advanced the knowledge base of highway runoff quality. The FHWA and USGS recently prepared a three volume synthesis of national highway runoff water quality data entitled The National Highway Runoff Water-Quality Data and Methodology Synthesis (Granato et al., 2003a, 2003b, 2003c). This project involved assembling, evaluating, and cataloging existing information, including existing data, and studies from the FHWA, the USGS, State DOTs, and other sources. Figure 3-7 shows the distribution of these highway runoff studies according to the 15 climatic regions identified by Driscoll et al. (1989). This figure along with the FHWA online searchable database (http://ma.water.usgs.gov/fhwa/biblio/default.htm) can be used to help identify the availability of highway runoff quality data within various United States climatic regions.

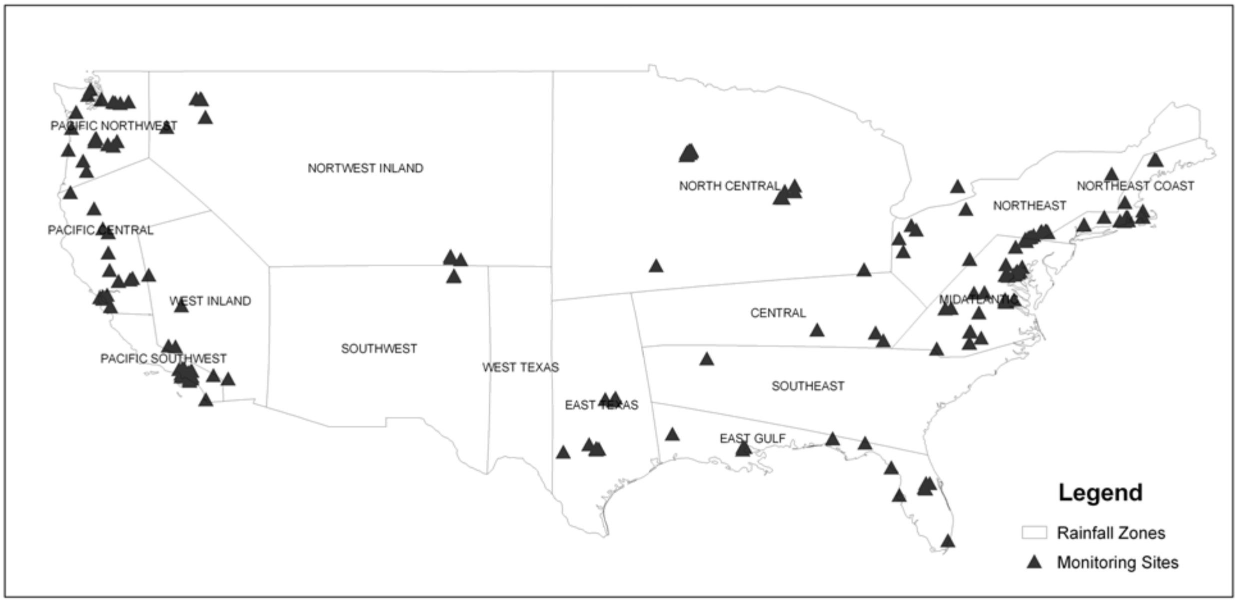

Figure 3-7. Highway runoff water quality studies throughout the continental United States.

In the absence of site-specific or region-specific highway runoff data, water quality can be estimated from average values reported in the literature. One of the more comprehensive highway runoff quality syntheses was conducted by Barrett et al. (1995b). Table 3-1 provides the ranges of average concentrations and loads reported in this synthesis. Note that some of these ranges are quite large. Therefore, the selection of values within these ranges must be based on a rudimentary knowledge of pollutant fate and transport, site-specific factors influencing pollutant loads and concentrations, and engineering judgment. The following subsections discuss typical characteristics of highway runoff pollutants and factors that may influence runoff quality. Additional information on highway runoff pollutants, including treatability information can be found in the pollutant fact sheets in Appendix A.

Table 3-1. Ranges of average values of common highway runoff constituents.

Constituent	Concentration (mg/L unless indicated)	Load (kg/ha/year)	Load (kg/ha/event)
SOLIDS			
Total	437-1147		58.2
Dissolved	356	148	
Suspended	45-798	314-11,862	1.84-107.6
Volatile, dissolved	131		
Volatile, suspended	4.3-79	45-961	0.89-28.4
Volatile, total	57-242	179-2518	10.5
METALS (totals)			
Zn	0.056-0.929	0.22-10.40	0.004-0.025
Cd	ND-0.04	0.0072-0.037	0.002
As	0.058		
Ni	0.053	0.07	
Cu	0.022-7.033	0.030-4.67	0.0063
Fe	2.429-10.3	4.37-28.81	0.56
Pb	0.073-1.78	0.08-21.2	0.008-0.22
Cr	ND-0.04	0.012-0.10	0.0031
Mg	1.062		
Hg x 10^{-3}	3.22	0.007	0.0007
NUTRIENTS			
Ammonia, total as N	0.07-0.22	1.03-4.60	
Nitrite, total as N	0.013-0.25		
Nitrate, total as N	0.306-1.4		
Nitrite+nitrate	0.15-1.636	0.8-8.00	0.078
Organic N, total as N	0.965-2.3		
TKN	0.335-55.0	1.66-31.95	0.17
Nitrogen, total as N	4.1	9.80	0.02-0.32
Phosphorus, total as P	0.113-0.998	0.6-8.23	
MISCELLANEOUS			
Total coliform (no./100mL)	570-6200		
Fecal coliform (no./100mL)	50-590		
Sodium		1.95	
Chloride		4.63-1344	
pH	7.1-7.2		
Total Organic Carbon	24-77	31.3-342.1	0.88-2.35
Chemical Oxygen Demand	14.7-272	128-3868	2.90-66.9
Biological Oxygen Demand (5-day)	12.7-37	30.60-164	0.98
Polyaromatic Hydrocarbons (PAHs)		0.005-0.018	
Oil and Grease	2.7-27	4.85-767	0.09-0.16
Specific conductance (µS at 25^{o}C)	337-500		
Turbidity (JTU)	84-127		
Turbidity (NTU)	19		

Source: Barrett et al. (1995b).

3.5.1.1 Solids and Sediment

In highway environments, pavement, tire, and vehicular abrasion can be major sources of solids. Solids may range in size from soluble, submicron particles to insoluble gravel size aggregate (Sansalone and Buchberger, 1997). Particle size influences the transport and treatability of solids in stormwater runoff. Sansalone et al. (1998) found that particle counts and diameters in runoff were strongly influenced by flow intensity (especially at higher flows) and thereby also by traffic intensity (discussed more in Section 3.5.2). Smaller particles (< 8 um) were rapidly washed off during high flows, and even when the high flows had relatively short durations, a mass-limited condition was observed (i.e., concentrations diminished prior to the decline of flow rate). This observation indicates that highway solids exhibit first flush characteristics, particularly during higher intensity storms. However Sansalone et al. (1997, 1998) found that dissolved solids typically exhibit a stronger first flush than suspended solids regardless of rainfall intensity or flow, but total solids typically exhibit a weaker first flush during low rather than high flow rate events.

3.5.1.2 Metals

Compared to typical domestic wastewater and receiving water discharge criteria, metal species concentration in highway runoff can exhibit significantly elevated levels on an event basis as showed in Table 3-2 (Sansalone and Buchberger, 1997; Drapper et al., 2000; Barrett et al., 1995a, 1995b; Driscoll et al., 1990).

Table 3-2. Metals concentrations in urban highway runoff.

Parameter	Units	Urban highway runoff			
		Sansalone and Buchberger (1997)[1] Median (Range)	Drapper et al. (2000) (Range)	Barrett et al., (1995a, b) (Range)	Driscoll et al., (1990) (Median)
TSS	ug/L	750 (150-22000)	60-1350	19-798	142
Total Zn	ug/L	628 (336-15244)	150-1850	22-929	330
Dissolved Zn	ug/L	1322 (209-14786)	---	---	---
Total Cu	ug/L	88 (43-325)	30-340	22-7033	50
Dissolved Cu	ug/L	44 (13-279)	---	---	---
Total Pb	ug/L	88 (31-1457)	80-620	7-1780	400
Dissolved Pb	ug/L	16 (13-21)	---	---	---
Total Cd	ug/L	8 (5-32)	---	---	---
Dissolved Cd	ug/L	3 (2-9)	---	---	---

1: The concentration is event mean concentration (EMC)
2: the total metal discharge criteria are State EPA criteria for discharge to modified warm water surface water, and the dissolve metal discharge criteria are USEPA criteria for discharge to modified warm water surface water

Runoff from roadways transports dissolved, colloidal and suspended solids in a heterogeneous mixture that includes inorganic and organic anthropogenic constituents. Parameters such as pH, alkalinity, traffic levels, entrained solids characteristics and residence time have been shown to influence the partitioning of metal species (Sansalone and Buchberger, 1997). Results from partitioning analysis between the dissolved and particulate-bound fractions on a mass basis indicate that Zn, Cd, Cu and Pb can be predominately dissolved in urban pavement sheet flow for residence times less than an hour at runoff pH levels between 6 and 7.5 and rainfall pH levels less than 4 (Sansalone and Glenn, 2000). Recent research on highway stormwater runoff in Cincinnati indicates that dissolved fractions for Cd, Cu and Zn typically exceed 70%, while Pb exceeds 50% based on partitioning data for lateral pavement sheet flow (Sansalone and Buchberger, 1997). Urban stormwater runoff characterization in urban residential areas of London showed dissolved fractions for Cd of 69%, Cu of 87%, Pb of 47% and Zn of 82% (Revitt et al., 1990). The deposition and accumulation of these dissolved and particulate-bound constituents result from traffic activities, vehicular component wear, fluid leakage, pavement degradation, leaching and roadway maintenance (Armstrong, 1994; Ball et al., 1991; Lygren et al., 1984; Muschack, 1990).

From a study conducted on an urban Cincinnati highway site, both the dissolved and particulate fractions of metal elements were measured, and it was determined that rainfall events with lower pH and higher average residence times generally resulted in significantly higher dissolved metal fractions (Sansalone and Buchberger, 1997). The low alkalinity of the asphalt pavement was found to have little effect on the pH of the runoff, thus not significantly neutralizing the runoff and affecting the partitioning, whereas the Portland cement concrete pavement was hypothesized to provide the alkalinity necessary to significantly raise the pH (Sansalone and Buchberger, 1997). In addition, it was found that metal elements such as lead, iron, aluminum, and chromium tended to be bound as particulates, while zinc, cadmium, and copper were predominately found in their dissolved form (Sansalone and Buchberger, 1997). However, other authors (Yonge et al., 2002) have found zinc primarily in the particulate-bound phase. Related to partitioning Glenn et al. (2002) found that dissolved mass, even for relatively insoluble metals, dominates particulate-bound mass at the edge of a highway shoulder, while major peaks in a runoff hydrograph tend to correspond to a decrease in the dissolved heavy metal mass, resulting from increased partitioning to solids mobilized by higher flows.

3.5.1.3 Nutrients

Generally nitrogen and phosphorus are the primary nutrients of concern because of their ability to enhance algal production, thus reducing the oxygen concentration in a receiving water body. Nitrogen and phosphorus in stormwater are generally a result of runoff of lawn fertilizers, atmospheric fallout, and discharges from automobile exhaust and other combustion processes (Strecker, 1994). Generally, three forms of nitrogen are measured and analyzed when looking at stormwater runoff: inorganic nitrogen (nitrite and nitrate), ammonia nitrogen, and total Kjeldahl nitrogen (TKN), which is the sum of ammonia and organic nitrogen.

Three forms of phosphorus are also typically measured and analyzed: ortho-phosphate, which is the most biologically available, soluble phosphate (ortho-phosphate and organic phosphorus), and total phosphorus (Strecker, 1994). Total phosphorus and ortho-phosphate are the typical forms included in monitoring regimes because they characterize both the total and bio-available forms, and like nitrogen, are generally reported in terms of their mass of phosphorus (Strecker, 1994).

In urban runoff, the first flush phenomenon is often observed for nutrients, although with a much more variable pattern than for metals and solids, due to the higher speciation frequency and the less stable nutrient forms observed. Nitrate and ortho-phosphate were rarely observed by Yonge et al. (2002) during first-flush events and did not show significant strength, as ortho-phosphate was observed in only 18% of analyzed events. However, TKN, ammonia, and total phosphorus were readily observed in the first flush events, with a relative strength in the order of TKN and ammonia > nitrate.

3.5.1.4   Petroleum Compounds

In roadway environments, oil and grease are prevalent organic constituents but are rarely measured due to difficulties in obtaining accurate, uniform samples. Oil and grease are generally present in five forms, only three of which are removable by flotation - free oil, dispersed or emulsified oil, and sorbed oil. The two forms that are not removable are oil that has been stabilized by a surfactant like soap or detergent and dissolved oil (Minton, 2005). The free and dispersed oils exist as minute droplets about 10-100 microns in size, which may be sorbed to suspended solids or exist independently and produce a sheen that many jurisdictions do not allow (Minton, 2005). Generally, flotation and sedimentation represent the primary processes for removal. Typical mean concentrations in stormwater range from 0.57 to 69 mg/L and are generally the result of vehicular exhaust and lubricating oils (Minton, 2005).

Several factors influence the relative concentrations of hydrocarbons in an urban environment, including precipitation, land use, traffic volume, and population (Hoffman and Quinn, 1987). Studies by Hoffman and Quinn (1987) and Hoffman et al. (1985) indicate that a majority of hydrocarbons in urban runoff are associated with particulate matter, regardless of land use, and partitioning between the particulate and soluble phases is most typically a function of chemical properties, temperature, and time in solution. Particulate hydrocarbons are generally called aliphatic hydrocarbons and consist of normal and branched alkanes and cycloalkanes (Meyers, 1987).

While a majority of treatable petroleum hydrocarbons are sorbed onto sediment due to their hydrophobic nature and high sorption coefficients, some also remain in solution, typically the less dense gasoline products (Minton, 2005). This partitioning affects the relative wash-off characteristics of petroleum hydrocarbons, although first flush characteristics of petroleum hydrocarbons, regardless of partitioning, have been found to display similar peaks to those of solids and metals, and increase according to the relative flow rate (Hoffman et al., 1985). The concentration of petroleum hydrocarbons during secondary flush situations tends to be more pronounced compared to metals and solids, indicating that thorough transmittal of hydrocarbons during first flush events does not readily occur, although eventually the source will be depleted and concentrations will significantly taper (Hoffman and Quinn, 1987). First flush characteristics and patterns for petroleum hydrocarbons have been observed to differ, compared to other pollutants, under snowmelt rather than rainfall events, for most pollutants generally exhibit lower loadings for snowmelt situations compared to rainfall, but hydrocarbons averaged 30% greater for snowmelt (Hoffman and Quinn, 1987).

Removal of hydrocarbons has been observed biotically under aerobic conditions, but typical removal practices involve flotation and sedimentation systems. Oil-water separators, detention ponds and sand filters have all been used as removal methodologies for the particulate based hydrocarbons. Biotic removal processes are found to most readily address the soluble, aromatic compounds with low atomic weights, and generally a high nutrient status and an aerated environment are the most important factors driving biological degradation (Fought and Westlake, 1987).

A group of toxic and carcinogenic compounds rarely included in characterization studies, but often present in highway runoff due to traffic-related sources, consists of polycyclic aromatic hydrocarbons (PAHs). PAHs represent over 2000 PAH compounds; only sixteen have been placed on the EPA list of priority pollutants (Pawluk et al., 2002). PAHs are ubiquitous, and are emitted from practically every combustion source. Following combustion PAHs enter the atmosphere and subsequently rivers and lakes through wet deposition, or dry deposition where they are later washed away by stormwater runoff. Specific traffic-related sources of PAHs include tire wear, asphalt and asphalt coatings, vehicle exhausts, and lubricating oils and grease (Pawluk et al., 2002). Other sources include industrial effluents and spills or intentional dumping.

Some PAHs will evaporate from water and soil, but the majority of PAHs in stormwater are usually found in particulate form. A stormwater runoff study done by Marsalek et al. (1997) found that the dissolved phase PAHs represented less than 11 percent of the total concentrations. In another study, Shinya et al. (2000) found that the higher molecular weight PAHs were more associated with suspended solids in the runoff and the predominant PAHs (phenanthrene, fluoranthene and pyrene) comprised about 50 percent of fifteen quantified PAHs constituents in each sample. In results from Ames' assay, mutagenicity was appreciably associated with PAHs in the particulate fraction of runoff water. The dissolved fraction also showed positive mutagenic response by unknown soluble aromatic compounds.

3.5.1.5 Additional Potential Constituents of Concern in the Highway Environment

Volatile Organic Carbons (VOCs)

Many studies have been conducted since 1970 to characterize concentrations of semivolatile organic compounds (SVOCs) in highway runoff and urban stormwater (Lopes and Dionne, 1998). Suspended solids appear to be the most significant factor affecting SVOC concentrations in water. In sediment, the most significant factors affecting SVOC concentrations are organic carbon content and distance from sources such as highways and power plants (Lopes and Dionne, 1998). Petroleum hydrocarbons, oil and grease, and polycyclic aromatic hydrocarbons (PAHs) in crankcase oil and vehicle emissions are the major contributors of SVOCs in highway runoff and urban stormwater.

Urban land surfaces are the primary nonpoint source of VOCs in stormwater, but concentrations are very temperature dependent (Lopes and Dionne, 1998). The atmosphere is another potential source of hydrophilic VOCs in stormwater, especially during cold seasons when partitioning of VOCs from air into water is greatest. While limited treatability data are available for VOCs and SVOCs from stormwater, their physical characteristics indicate that these constituents could be treated by maximizing volatilization (see Section 4.2.4) and sedimentation (Section 4.2.3).

Methyl tertiary-butyl ether (MTBE), a gasoline oxygenate, disperses rapidly in water and is less biodegradable than common gasoline compounds, such as benzene, toluene, ethylbenzene, and total xylene (BTEX). The U.S. Geological Survey (USGS) sampled stormwater in 16 cities and metropolitan areas that were required to obtain permits to discharge stormwater from their municipal storm-sewer system into surface water (Delzer et al., 1996). Concentrations of 62 VOCs, including MTBE and BTEX compounds, were measured in 592 stormwater samples collected in these cities and metropolitan areas from 1991 through 1995. MTBE was the seventh most frequently detected VOC in urban stormwater, following toluene, total xylene, chloroform, total trimethylbenzene, tetrachloroethene, and naphthalene. Toluene and total xylene were the most frequently detected BTEX compounds and the most frequently detected VOCs in these investigations.

Platinum Group Metals (PGMs)

Another group of elements that are more recently being found in urban and highway runoff are the platinum group metals (PGMs): palladium, platinum, and rhodium. PGMs are used in catalytic converters to abate the emission of aromatic hydrocarbons, CO and NOx. Due to the thermal and mechanical conditions under which autocatalysts work (including abrasion effects and hot-temperature chemical reactions with oil fumes), they can cause significant release of the PGMs to the environment in the form of fine particles (Carolia et al., 2000). This raises concern because platinum is a known cytotoxin and tends to bioaccumulate. Due to air quality regulations requiring catalytic converters in all new cars, the amount of PGMs released into the environment each year is expected to continue to increase. A study conducted by Rauch et al. (1999) investigated the concentrations of PGMs in road sediment samples in 1984, 1991, and 1999 and found a clearly increasing trend, especially for particles smaller than 63 μm.

Potassium Perchlorate

Perchlorate is an inorganic anion and oxidant consisting of chlorine bonded to four oxygen atoms ($ClO4-$). It is typically found in association with ammonium, sodium, or potassium cations as a salt. Perchlorate is a known soil and groundwater contaminant with potentially significant health implications. Currently there are no federal drinking water standards for perchlorate, but some states have adopted public health goals for drinking water supplies at the quantitative detection level of 4 ppb (OEHHA, 2006).

Potassium perchlorate is a primary ingredient used in fireworks and safety flares. Safety flares are used in emergency situations for road-side accidents and rail and marine emergencies. Surface runoff from highways and roads represents a potentially significant and largely uninvestigated impact to surface water and groundwater quality. Preliminary research by Silva (2003a, 2003b) of the Santa Clara Valley Water District in California indicates that a 20-minute road flare may leach 1.95 mg of perchlorate if completely burned compared to 3,645 mg of perchlorate if unburned, but damaged (i.e., run over by a car). Since extremely low concentrations of perchlorate may contaminate groundwater supplies, portions of highways with high road flare use due to frequent vehicle breakdowns or accidents may be an environmental impact concern. Additional evaluation of the potential for perchlorate impacts to surface waters and groundwater from safety flare use appears to be warranted.

3.5.2 Factors Affecting Runoff Quality

Many anthropogenic factors may affect highway runoff quality including: traffic density, surrounding land uses, driving activity, road maintenance practices, and construction and repair materials. However, rainfall and runoff studies that have attempted to identify a relationship between any of these variables and pollutant concentrations are generally inconclusive. For many pollutants, the mass of pollutant in the runoff is more important for estimating the impacts of runoff on receiving water quality than is the instantaneous concentration. The pollutant load is largely a function of the volume of runoff rather than the concentration and is generally predicted more accurately.

This section provides a brief overview of the anthropogenic factors that adversely affect runoff quality. For each of the sub-topics presented below, a description of the type of constituents generated from each activity and the potential impacts of constituents to receiving water is discussed. This section provides a foundation for identifying the constituents that may be present at a site based on the activities that may be occurring at the site.

3.5.2.1 Traffic Density and Land Use

The distinction between urban and rural areas directly relates to the probable pedestrian and traffic volumes observed at a site. Some studies (Yousef, 1985; Driscoll et al., 1990; Barrett et al., 1995a) have used average daily traffic (ADT) as an indicator of traffic density, while others (Racin et al., 1982; Chui et al., 1982; Kerri et al., 1985, and Sansalone et al., 1998) have investigated cumulative vehicles during a storm (VDS). Driscoll et al. (1990) found that there was no definitive relationship between traffic density and pollutant levels. However, Barrett et al. (1995a) found higher concentrations of all monitored constituents at the higher traffic volume sites, even when normalized for surface area.

In a study funded by the Texas Department of Transportation (TxDOT), water quality of highway runoff in the Austin, Texas area was determined by monitoring runoff at three locations on the MoPac expressway, which represented different daily traffic volumes, surrounding land uses, and highway drainage system types (Barrett et al., 1995b). The highest concentrations of all constituents were measured at the high traffic site (> 30,000 average daily traffic). The concentrations at all sites were similar to median values for similar sites compiled in the FHWA nationwide studies of highway runoff quality.

Sansalone et al. (1998) found that low runoff volume events (in high traffic situations) typically exhibit a wash-off trend that is flow limited (more solids available for discharge than could be washed off), but reducing the preceding dry days had a lowering effect on the amount of solids available for discharge and showed a trend similar to the high runoff volume events. In comparison, high runoff volume events with low traffic situations show that wash-off of solids is mass limited (fewer solids available for further discharge) (Sansalone et al., 1998). Overall, these studies indicate the influence of vehicular traffic when looking at mass transport over a site.

In an analysis of data from the extensive 1997-2001 Caltrans highway runoff characterization study, Kayhanian et al. (2003) found that, in general, EMCs from urban highways were greater than EMCs from non-urban highways, with the exception of TSS, COD (chemical oxygen demand), TDS (total dissolved solids), turbidity, NH3, and diazinon, for which average EMCs were higher on non-urban highways. The authors suggest that other than transportation-related sources must contribute to this latter group of constituents. Interestingly, no simple linear correlation was found between annual average daily traffic (AADT) and EMCs, including those known to be related to transportation activities (e.g., Pb, Cu, Zn, and oil and grease). However, AADT did contribute to significant multiple regression relationships when combined with other factors such as antecedent dry period, seasonal cumulative rainfall, total event rainfall, maximum rainfall intensity, drainage area, and land use. (The latter three were generally the least important factors in the multiple regressions.) The multiple regression relationships developed by the authors are useful for EMC estimates based on these several independent variables, with the caveat that the data are heavily influenced by the majority of monitoring sites located in Southern California.

3.5.2.2 Acceleration/Deceleration Areas

Limited data are available to assess whether vehicle acceleration and deceleration influence pollutant build-up and washoff from roadways. However, since common stormwater pollutants are known to exist in tires, brake pads, and exhaust there is a reason to believe that acceleration and deceleration areas such as on-ramps and off-ramps may contribute higher proportions of certain pollutants than other highway areas. Drapper et al. (2000) noted that highway runoff monitoring sites with exit lanes had higher concentrations of acid-extractable copper and zinc, supporting the hypothesis that brake pad and tire wear caused by rapid deceleration may impact water quality. However, this observation could not be statistically verified due to the small number of observations.

3.5.2.3 Road Maintenance Practices

Street cleaning falls into two categories, flushing and sweeping. Street flushing cleans a larger street area and is capable of picking up fine particulates, but is most appropriate for streets served by combined sewers so the wash water is not discharged to receiving waters. Street sweeping is more appropriate for areas served by storm sewers and is more prevalent than flushing in the United States.

Performance monitoring studies conducted as part of The Nationwide Urban Runoff Program (NURP) concluded that street sweepers were not very effective in reducing pollutant loads (USEPA, 1983). The mechanical street sweepers of that era were ineffective for reducing roadway pollutant sources because they were unable to pick up fine-grained sediments that carry a substantial portion of the pollutant load. Two types of street sweeper technology, regenerative-air and vacuum-assisted sweeping, are now available that are capable of improved performance over the mechanical sweepers. Regenerative-air sweepers use air to loosen particles, while the vacuum assisted sweepers use a dry broom to loosen particles prior to vacuuming into the hopper. Regenerative-air sweepers remove a higher fraction of street sediments that mechanical sweeping alone, while vacuum assisted sweepers generally result in the best overall removal of sediments (CWP, 1999).

The overall effectiveness of a street sweeping program is dependant on many factors besides the effectiveness of the sweepers. The frequency of sweeping, rate of pollutant accumulation, accessibility to curbs (i.e., parked cars limit access), and amount of coverage will all affect the potential effectiveness of a street sweeping program. Of course, rainfall characteristics of the location play an important role as well. For example, in an area with frequent winter storms, street sweeping may not be practical due to insufficient time between storms to make sweeping worthwhile in the wet season, but may be effective during dryer months.

Refer to Section 11 - Pollution Prevention/Street Sweeping of the *LID Design Manual* for additional information on street sweeping.

3.5.2.4 Construction and Repair Materials

Roads are a potential source of pollutants in runoff due to transportation activities and from activities involving construction of new road networks and continual rehabilitation and maintenance of the existing networks. Construction and maintenance of our system of highways relies on the use of a wide variety of materials, including Portland cement, asphalt cement, petroleum-base sealants, wood preservatives, and various additives. During wet weather, there is potential leaching of the chemical constituents in these materials and the possibility of transport to adjacent surface and subsurface water bodies.

A study by NCHRP (Nelson et al., 2001), identified potentially mobile constituents from a wide range of highway construction and repair (C&R) materials and measured their potential impact on surface water and groundwater. The materials included conventional, recycled, or waste products, but excluded constituents originating from construction processes, vehicle operation, maintenance operations, and atmospheric deposition. Construction and repair materials have historically been viewed as innocuous and hence not of concern to environmental quality; however, there are legitimate questions about the impact of some of these materials on the environment. Furthermore, recycled and waste materials are increasingly being promoted as environmentally friendly substitutes for conventional construction and repair materials, thereby increasing the number of nontraditional materials in contact with surface and ground waters. Some of the waste and byproduct materials used for highway construction and repair include scrap tires, coal ash, foundry sand, municipal solid waste, phosphogypsum, and recycled asphalt.

The NCHRP study (Nelson et al., 2001) indicated that in their "raw" form, that is, prior to incorporation into an "assemblage" such as asphalt concrete (AC) mix or Portland cement concrete (PCC), many highway materials apparently exhibited toxicity to algae (*Selenastrum capricornutum)* and daphnia (*Daphnia magna*), the target aquatic organisms selected for use in the project. However, for most of the materials, toxicity is considerably reduced after incorporation into the final assemblage (e.g., pavement or fill). Study results also indicated that the observed toxic effects to aquatic organisms are generally much lower under field conditions because of mass transfer effects. Most material leachate toxicity observed in aquatic organisms is either reduced or eliminated by sorption onto soils and through other degradation processes such as volatilization, photolysis, and biodegradation. Moreover, it is expected that the sorption onto soils in the highway environment will be much greater than that exhibited in these laboratory tests because of the much higher solid: liquid ratios under field conditions.

Overall, it is likely that that most C&R materials behave in a benign fashion in the environment. On the highway surface, leaching is slow, transport is rapid, and dilution is large – all leading to very low initial concentrations of potential contaminants. Furthermore, rainfall is intermittent and leaching rates decrease with time. Transport in soils is generally very slow owing to high compaction and low hydraulic conductivities. Thus, vertical water movement in the highway base is slow, in the range of meters per year. Vertical migration of contaminants is even slower because of the high sorption capacity of most soils. Many model tests showed vertical migration of contaminants on the order of only few millimeters over a simulation period of many days.

In rare cases, some highway construction and repair materials have the potential to generate runoff with appreciable toxicity. However, toxicity resulting from leaching of the construction or repair material will decrease with time, so these materials will generally contribute to pollutant loads in runoff only over the short term. Therefore priority should be given to pollutants that are expected to occur from the highway environment in the long term when selecting BMPs for treatment systems. Careful consideration should be given to the use of potentially toxic construction and repair materials in sensitive environmental areas.

3.5.3 Identification and Prioritization of Pollutants of Concern

After the water quality at a site has been characterized to an acceptable extent based on available information, the pollutants of concern should be identified and prioritized. If existing data at a site are not readily available, then site characteristics should be used to estimate the types and levels of pollutants likely to be present. For example, if a particular section of highway receives high average daily traffic and is located in a highly urbanized area, average storm event concentrations of copper and zinc are likely at the upper end of the ranges shown in Table 3-2 and likely represent constituents of concern. Once pollutants of concern have been identified, they can be prioritized as to minimize the number of available treatment alternatives for a given set of site conditions and constraints. The following criteria should be used to help prioritize the pollutants of concern:

- **High Priority:** The pollutant is present or expected to be present at levels of concern AND is regulated in the discharge or receiving water body OR the pollutant will impede, obstruct, or impair the treatment mechanisms that reduce another high priority pollutant.
- **Medium Priority**: The pollutant is present or expected to be present at levels of concern, but is NOT regulated in the discharge or receiving water body.
- **Low Priority**: The pollutant is more effectively addressed by elimination of the pollutant source OR the available treatment technologies are not practicable for addressing the pollutant.

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CHAPTER 4 FUNDAMENTAL PROCESS CATEGORIES AND LID PRINCIPLES

After site conditions, constraints, influent water quality and hydrology, have been determined and/or estimated, the unit processes that are available to reduce or treat the pollutants of concern should be identified and qualitatively ranked on a relative scale according to how well those processes reduce runoff volumes/rates or treat the pollutants of concern. Some of the unit processes discussed in this chapter may not be common in highway runoff treatment applications. However, as more advanced stormwater treatment systems are required to protect sensitive receiving waters, the highway practitioner may find a need to utilize alternative unit processes in their BMP designs. For example, the California Department of Transportation has been researching rare and novel methods for removing very fine sediment (<20μm) from their highways in the Lake Tahoe basin in response to decreasing clarity of the lake and the ensuing fine sediment TMDL (Regenmorter et al., 2002; Caltrans, 2003). The following sections provide a conceptual review of unit operations and processes (UOPs) potentially needed to treat highway runoff to a target level.

4.1 Hydrologic/Hydraulic Operations

Flow alteration is a significant unit operation for stormwater runoff; and historically has been the single major unit operation for storm water management for decades in the USA. In large part, flow alteration is implemented as a hydrologic control. Flow alteration and water quality changes are linked and therefore flow alteration translates to water quality alteration. Flow alteration includes modifications to components of the hydrologic cycle such as infiltration, detention, storage and evaporation. In general the goals of these physical operations (recognized as hydrologic controls) have been to reduce volume, reduce peak flows, generate steadier flows in time, and attenuate temporal aspects of flow. To varying degrees these hydrologic controls can have a significant impact on water quality. Applications of hydrologic modification are ubiquitous in the built environment and are intentional or inadvertent, as well as beneficial or detrimental. Examples of intentional applications that have potential water quality and quantity benefits include infiltration, detention and flow equalization, while detrimental applications include impervious paving or loss of vegetation.

The underlying principles of flow attenuation and volume reduction are based primarily on the elements of the hydrologic cycle, including infiltration, evapotranspiration, interception, conveyance, detention, and retention. The following subsections discuss three common hydrologic control unit operations: peak attenuation, volume reduction or minimization of volume increases, and flow duration.

4.1.1 Flow Attenuation

Flow attenuation refers to hydrologic operations responsible for reducing peak event discharges (e.g., peak shaving). The primary mechanisms involved in flow attenuation include interception, conveyance and detention, and to a lesser degree infiltration.

4.1.1.1 Interception

Interception is a form of detention and retention storage that occurs when leaves, stems, branches, and leaf litter catch rainfall. Interception is considered to be detention storage if raindrops drain off vegetation by "through-fall" (dripping off a leaf onto the ground) or by stem flow (flowing down stems or trunks). Vegetative through-fall accounts for the majority of the movement of intercepted rainfall. Intercepted rainfall that is retained is lost to the atmosphere by evaporation from the surface of leaves.

The percentage of rainfall that is intercepted increases with the density of vegetation, including all vertical layers from canopy to leaf litter. At maximum density, both trees and grasses may intercept 10 to 20% of precipitation from an individual storm. Per unit of ground area, some grass species have the same leaf area as many trees (Dunne and Leopold, 1978).

Volume reductions from interception should be expected to be minor in urban BMPs. However, interception also absorbs some of the kinetic energy of rainfall; this reduces the magnitude of raindrop erosion. Similarly, interception preserves soil permeability because less eroded material is available to clog soil pores and less soil compaction by raindrops occurs.

4.1.1.2 Conveyance

Conveyance is the transport of surface runoff and includes the entire flow path from where a raindrop falls to where it enters the receiving body of water. In conventional stormwater designs, conveyance is synonymous with the efficient drainage of runoff. By contrast, decentralized controls that provide conveyance also promote infiltration, improve water quality, and increase runoff travel time, or time of concentration (Tc). They are often critical components of the treatment train approach. In this report, “conveyance” refers to the act of transporting runoff, rather than the carrying capacity of a BMP or other structure.

4.1.1.3 Detention

Detention is the temporary storage of stormwater, which is released over a period of hours or days after rainfall ceases. Detained stormwater may exist as ponded free water or can be held within moist soil. In highly urbanized highway environments, detained runoff ultimately enters the storm drain system. In a vegetated BMP, ponded water and any soil moisture above the field capacity are detained, rather than retained, because that portion of the stormwater slowly percolates by gravity through the soil column into the underdrain. For small, frequently-occurring storms, however, the release of detained water will not usually cause flooding because the stormwater will enter the system over a much longer period of time, and at a lower rate, than if decentralized controls were not in place.

4.1.2 Volume Reduction / Minimization of Volume Increases

These are the unit operations responsible for reducing the total volume of runoff, such as infiltration and evapotranspiration. Runoff can also be captured and reused (e.g., irrigation water) in BMPs such as underground tanks and vaults. If pollutant loads are a high concern, identify volume reduction unit operations that will reduce the total runoff volume to the receiving waters.

Site design principles can be employed to minimize increase in runoff volumes, e.g., draining roofs and driveways to permeable / landscaped areas, permeable pavements for parking.

4.1.2.1 Retention

Retention captures stormwater permanently. The volume of retained runoff never enters the storm drain system; vegetative interception, evaporation, transpiration of soil moisture, and reuse eliminate this volume. Evaporation may occur from soil, vegetation, or hard surfaces. Transpiration reduces the water volume within the root zone of soil. As stormwater enters a BMP, infiltrating water will be retained up to the point that the soil moisture content equals the field capacity. If the rainfall is sufficiently light such that the soil moisture content in a vegetated BMP never reaches field capacity, ET alone will eliminate the volume of stormwater in the soil.

4.1.2.2 Infiltration

Infiltration is the downward movement of water into the soil via percolation through pore spaces. In an open system such as a meadow, this movement is unrestricted and water can infiltrate down to, and recharge, the groundwater table. Groundwater recharge is a basic component of the natural hydrologic cycle. In urban areas, unrestricted infiltration may exacerbate infiltration and inflow (I/I) problems in both separate and combined sewer systems; the likelihood of this scenario must be evaluated before constructing unlined infiltration BMPs.

In urban highway BMPs, infiltration often occurs in a closed system. Functionally impervious (i.e., compacted) soils or an impermeable liner limit the downward movement of water. (Designs often include underdrains and high flow bypass pipes to ensure adequate drainage.) Both closed and open infiltration BMPs contain one or more layers of engineered soil media ranging in thickness from a few inches (as in a green roof) to several feet. Two basic processes occur in these layers:

- Volume reduction through the filling of soil pores.
- Pollutant removal through filtering, and sorption by organic matter and other soil constituents.

In urban areas, some of the infiltrated stormwater will be retained and its volume permanently taken “out of the system.” Stormwater may also be detained, which temporarily reduces the amount of stormwater that would otherwise be in the storm drain system and allows it to enter the system over an extended period of time.

The soil moisture content determines the volumes of stormwater that are retained and detained. In a given BMP, the volume of retained water is the volume for which the soil moisture content equals the soil’s field capacity. The retained water leaves the soil through ET. The field capacity is the point at which free drainage by gravity ceases and the remaining water is held in the soil pores by capillary and osmotic forces. At this moisture content, the soil is unsaturated. The volume of additional stormwater that causes the soil moisture content to exceed the field capacity will be detained, and will drain by gravity into underdrains over a period of several hours or days. In this report, the capacity for groundwater recharge is not required for a BMP to be considered an infiltration device. Infiltration BMPs may be part of closed or open systems. In highly urbanized highway environments, open systems are rare.

4.1.2.3 Evapotranspiration

Evapotranspiration (ET) refers to the combined effects of evaporation and transpiration in reducing the volume of water in a vegetated area during a given time period of time (often daily or monthly). The volume of water in the root zone of soils is taken up by roots and then transpired by being diffused through leaves. Uptake by roots also removes a variety of pollutants from stormwater.

During the first two to three days after a rainfall, ponding and infiltration processes remove (i.e., detain) an appreciable fraction of the stormwater volume in a BMP, even though ET is occurring. After this time, gravitational drainage into the underdrains effectively ceases and the field capacity is reached. ET becomes the dominant process because the volume of water present in the soil at field capacity will be lost to the atmosphere through ET alone. Equation 4-1 gives the maximum volume of water that ET can potentially remove once the soil moisture content equals the field capacity (FC).

$$\text{Vol. transpired} = (\text{rooting depth} \times \text{soil surface area}) \times (\text{FC} - \text{wilting point}) \qquad [4\text{-}1]$$

The wilting point is the soil moisture content beyond which plants cannot exert enough suction to draw more water out of the soil. The difference between the field capacity and the wilting point is the moisture content available for transpiration.

The field capacity of urban BMPs can be designed to meet desired drainage characteristics. The BMP's connectivity to underlying soils, including the presence of underdrains and gravel bedding, also affects the field capacity. Many vegetated BMPs, such as rain gardens, have a low field capacity in order to maximize free drainage and filter pollutants.

4.1.3 Flow Duration

The flow duration method seeks to minimize the difference in both the magnitude of flows discharged from the watershed, and the cumulative time period each flow size persists between the pre- and post-development conditions. This approach incorporates principles from both peak attenuation and volume reduction, while taking into account the distribution of flows. Changes in flow duration are most easily detected from comparison of the pre- and post-development flow duration curve.

Comparing several different stormwater management strategies, including peak attenuation, volume reduction and flow duration matching, Palhegyi and Bicknell (2004) suggest that, of the three, flow duration offers the best means of limiting modification of downstream hydraulic conditions. While designing for peak flow attenuation over a range of design storm sizes accounts in part for potential impacts to downstream morphology from both large and small runoff events, it does not address the increased frequency of potentially erodible discharges. Volume reduction, in turn, fails to regulate the intensity of runoff from the watershed. By matching both the magnitude and frequency of flows under post-development conditions to those assumed for the pre-development watershed, flow duration helps govern the overall amount of "work" done on the downstream channel.

Flow duration is an appropriate strategy under most stormwater management conditions, while it is particularly applicable where impacts to the downstream channel or receiving waters are of paramount concern.

To achieve flow duration control, management systems must provide adequate regulation over the entire distribution of runoff flows and prevent significant increase in frequency of any given discharge. Where impoundments or basin-type stormwater controls are applied, flow duration control is most easily achieved through appropriate basin sizing and compound outlet design.

4.1.4 LID Principles

The overall objective of low-impact development (LID) is to minimize land development impacts on hydrology and water quality by using site design techniques and decentralized stormwater controls that maximize flow attenuation, volume reduction, and runoff duration and minimize the generation and transport of stormwater pollutants. The fundamental principles of LID provide a conceptual basis for evaluating, selecting, and integrating hydrologic and pollutant source controls into the highway environment. A detailed discussion of these principles, as well as LID design guidance, is provided in *The Low Impact Development Design Manual for*

Highway Runoff Control produced as part of this project. The paragraphs below briefly describe some of these fundamental LID principles.

4.1.4.1 Impervious Area Reduction and Disconnection

Impervious area reduction

Reducing the amount of imperviousness on the site will have a significant impact on the amount of compensatory centralized runoff control storage required since there is almost a one-to-one corresponding relationship between rainfall and runoff for impervious areas. For example, this could be achieved by building narrower shoulders and travel lanes if motorist safety can still be assured or restricting parking and sidewalk areas to one side of the road rather than both.

Impervious area disconnection

This technique involves avoidance of pipes and channels that directly convey stormwater from source to outlet by insertion of overland flow planes, small check dams, etc. These techniques can be effective at reducing energy, volume, peak, and maintaining natural flow paths. As described in the previous discussion of swales, many highway drainage systems that are already decentralized are still well connected to receiving waters. Often ditches are lined or armored with paved materials where alternative, more water quality friendly approaches may achieve the same erosion control and maintenance goals. Another advantage of disconnectivity in the highway environment is avoidance of piped drainage in cold climates. Frozen water in drainage systems creates a design and maintenance problem that can sometimes be avoided by discharge to open channels and pervious surfaces. Examples of impervious area disconnectivity would include use of permeable pavers for emergency stopping areas, crosswalks, sidewalks, road shoulders, on-street parking areas, vehicle crossovers and low-traffic roads. Also, directing impervious areas to sheet flow onto vegetated or bioretention areas to allow infiltration results in a direct reduction in runoff and corresponding storage volume requirements.

4.1.4.2 Minimize Development Impacts

Disturbance area reduction

Highway construction sites are common place in urbanized watersheds and could be a key source for releasing sediments, nutrients, and metals. Erosion and sediment control practices reduce sediment and associated pollutant loss to a certain extent during construction activities. Additionally, more control could be achieved by combining the above practices with practices for minimizing the disturbed area of a construction site. This would include among others preservation of infiltrable soils and preservation of existing natural vegetation.

Preservation of Infiltrable Soils

This approach includes site planning techniques such as minimizing disturbance of soils, particularly vegetated areas, with high infiltration rates (sandy and loamy soils), and placement of infrastructure and impervious areas such as roads and buildings on more impermeable soils (silty and clayey soils). Care must be taken when determining the suitability of soils for proposed construction practices. Adequate geotechnical information is required for planning practices. Plans should also be drawn to minimize or eliminate unnecessary movement of heavy machineries on the construction site that may compact the soil and reduce its infiltration capacity.

Preservation of Existing Natural Vegetation

Woods and other vegetated areas provide many opportunities for storage and infiltration of runoff. By maintaining the surface coverage to the greatest extent possible, the amount of runoff volume reduction by evapotranspiration is increased. Vegetated areas can also be used to provide surface roughness, thereby increasing the time of concentration. In addition, they function to filter out and uptake pollutants.

4.1.4.3 Integrated Management Practices and Conservation Design

Retro-fit of existing highway drainage facilities

Thousands of miles of existing highways are drained by roadside swales and pervious channels. These drainage facilities in many cases almost constitute "unintentional LID," in which the fundamental principles of infiltration, ET, and on-site retention of water need only be encouraged with minor design modifications, if any, to become bona fide LID facilities. Retrofit in the form of outlet controls (sizing of weirs, orifices, small check dams, etc.) may be enough for water quantity control (retention), and additional attention to vegetation may be enough to encourage better sedimentation and filtration of pollutants. It is important to utilize these opportunities for better management of runoff from existing roadways. For instance, replacing space gained by practices such as restricting parking and sidewalk areas to one side of the road rather than both with pervious drainage facilities such as vegetated channels would improve volume loss by infiltration.

Conserve natural topography and drainage courses

To the extent possible, the road design should be integrated into the natural surroundings by conserving natural drainage courses and aligning roads to follow existing topography and the sinuosity of rivers and shorelines. These principles serve both to improve the aesthetics of the roadway, as well as reduce the overall impacts on existing hydrology and geomorphology. Of course, if increases in flow frequencies or flow durations are expected, appropriate detention or energy dissipation methods must be used to protect the natural drainage courses downstream.

4.2 Physical Treatment Operations

A physical operation, in contrast to chemical or biological processes, is a form of treatment that is brought about by a physical mechanism, for example sedimentation. Historically, physical unit operations are some of the oldest mechanisms for improving water quality. Physical unit operations form the basis of many operations that are considered as preliminary and primary treatment in wastewater treatment. Physical unit operations are also the dominant forms of treatment in most so-called best management practices for stormwater runoff, whether these operations are an intentional or incidental design mechanism. Similar to Metcalf and Eddy (2003) these physical unit operations that treat particulate solids can be generally classified as:

1. Screening of solid material
2. Coarse solids reduction
3. Flow alteration (volume, peak, temporal attenuation) and equalization
4. Mixing and physical flocculation
5. Solids or grit removal
6. Sedimentation
7. Flotation
8. Filtration (depth, surficial, physical-chemical, membrane)

9. Clarification methods other than sedimentation
10. Solid separators such as accelerated gravity, deflective and membrane separators

The description of these unit operations is presented below as if each unit operation were a separate unit in a treatment process flow stream at a wastewater treatment plant. While this provides a good analog and method to explain the role of each unit operation it must be recognized that treatment of stormwater runoff is inherently different than centralized municipal wastewater treatment. Some stormwater runoff treatment systems developed around the concept of a treatment train approach similar to a wastewater treatment plant, but the majority of stormwater runoff treatment systems that are put in place are either single unit operations (possibly with some form of preliminary treatment) or multiple unit operations (intentionally or inadvertently) that are combined in a single structural system. Therefore, rather than presenting the following physical operations in the order of how they are typically found in a wastewater treatment plant, they have been categorized according to the physical characteristics of the operation.

4.2.1 Particle Size Alteration

Particle size alteration refers to both the reduction in size of coarse particles by shredding or grinding into smaller, more uniform sizes for subsequent removal by downstream unit operations and processes, as well as the increase in size of small particles through mixing and flocculation to be later removed via settling.

4.2.1.1 Coarse solids reduction

Coarse solids reduction includes methods of comminution, maceration and grinding. While these methods are standard practice in wastewater treatment, they are currently rare for treatment of stormwater runoff residuals. As centralized runoff control and treatment develops in conjunction with decentralized control, coarse solids reduction will be increasingly applied. The potentially hazardous waste designation for some stormwater runoff residuals will result in more advanced management of these residuals as compared to wastewater coarse solids. While solidification/stabilization of coarse solids has been investigated from a research perspective such management of coarse solids is still rare in practice.

4.2.1.2 Mixing and physical flocculation

Mixing in stormwater runoff is commonly carried out with the intention of blending or mixing one substance or phase with another or for the purpose of facilitating heat or mass transfer. While mixing encompasses a wide range of velocity gradients at a range of scales, mixing is generally classified as rapid (velocity gradient, $du/dz > 50\ sec^{-1}$) or slow mixing ($< 50\ sec^{-1}$). Rapid mixing residence times are generally less than 30 seconds while slow mixing such as for flocculation can range from several minutes to 1 hour. Mixing operations are typically applied in coagulation applications as rapid mixing to blend a chemical coagulant with entrained solids. Mixing operations are typically applied in flocculation applications as slow mixing to bring destabilized particles in contact with each other to promote floc growth. In slow mixing for flocculation, an equilibrium level is achieved between floc formation and floc breakup once the floc becomes too large and velocity gradients shear the large floc to a more moderate-sized floc that remains coherent when subject to shear forces from slow mixing and differential sedimentation.

Applications of flocculation (actually coagulation-flocculation) in stormwater runoff are becoming more common, but are still relatively rare. Further information on this chemical process is provided in 4.4.2.

4.2.2 Size Separation

Size separation describes the unit operations of screening and filtration that are used to remove gross pollutants from storm water. Flotation affects how gross solids are removed via size separation, but is more appropriately addressed as a density separation unit operations followed by a skimming operation, which is discussed in Section 4.2.3. Size separation is often used as a first line of defense to remove large objects at the early stages of the treatment cycle. Size separation provides for gross solids removal from stormwater by placing obstacles such as screens, racks and filters in the flow path that only allow the passage of particles that are smaller than the gaps in the obstacles.

Screening is used for removing large objects such as tree branches, household items, rocks and debris that may damage or obstruct downstream conveyance system elements such as pipes, pumps, and other BMPs. Screens are a physical exclusion mechanism intended to remove solids that have a dimension larger than a selected screen aperture. Coarse screens can have diameters as large as 150 mm in wastewater treatment and can be as small as 5 mm. Finer screens are generally not used for stormwater treatment due to the increased susceptibility to clogging.

Filtration can encompass a wide range of physical and chemical mechanisms, depending on the filter media; this discussion focuses on the dominant physical processes that typically occur in inert filter media. The materials removed by filtration may prevent drainage elements such as pumps, valves and pipes from clogging and other BMPs from being silted-up. Sansalone et al. (1998) observed that when the ratio of media diameter to particle diameter is less than 10, particles are usually removed by surficial straining. When the ratio is between 10 and 20, particles tend to undergo filtration within the pore volume, and this ratio range generally contributes to a loss of filtration capacity of the BMP if appropriate maintenance measures are not undertaken (Sansalone et al., 1998). Finally, with ratios greater than 20, little void space is filled by the particles and sedimentation and filtration tend to become the dominant removal processes.

In general, suspended and settleable solids concentration is brought to less than 50 mg/L through primary unit operations (e.g., screening and settling) before flow reaches a filter to ensure proper functioning of the filter. Filters are designed to remove particulate matter either on the surface of the filter or within the pore space of the filter. The build-up of such particles results in a significant increase of head loss; therefore, filters must be maintained by removing the accumulated particles on a regular basis. In wastewater plants this is carried out through regular backwashing. Filter maintenance is more challenging in stormwater runoff treatment systems, which tend to be passive, decentralized, and have far less oversight and monitoring than filters in wastewater treatment plants; yet they require similar or greater maintenance.

Size separation can often be an effective preliminary treatment for stormwater. Sites that have identified gross solids – both floatable and non-floatable – as constituents of concern are good candidates for size separation processes such as trash racks or collection devices at BMP inlets, e.g., retention ponds, and catch basins within storm drain inlets. Omitting size separation UOPs in treatment system design may negatively affect the performance of other unit treatment process, reduce the longevity of downstream BMPs, and increase maintenance frequency with little reduction in initial capital cost since size separation devices, particularly screening devices, are relatively inexpensive. Size separation through filtration such as a sand filter can provide the added benefit of removing stormwater constituents that may be attached to solids such as metals and bacteria. However, filtration unit processes have more stringent requirements for implementation including suspended and settleable solids concentrations amenable to filtration and more intensive maintenance requirements. A summary of the constituents that are effectively addressed through the use of size separation include: sediment (coarse and, to a limited degree, fine), floatables, trash, debris, and particulate-bound constituents such as fecal coliform and metals.

Size separation devices cause a head loss since they introduce obstacles in the flow path. Hence in situations where head losses are welcome such as pond inlets and outlets and stream inlets, size separation works well. Conveyance systems that have velocities that prevent deposition upstream of the size separation device may minimize head losses. Frequent maintenance also enhances size separation by reducing the obstacles in the flow path.

4.2.3 Density Separation

Density separation refers to the unit processes of sedimentation and flotation that are dependent on the density differences between the pollutant and the water to effect removal. Clarification is chemically enhanced sedimentation, but is not well suited to stormwater treatment so is not discussed further in this document. Flotation refers to the removal of floatables (e.g., trash, oil, etc.) often accomplished by forcing an influent stream beneath a boom, baffle or inverted outlet.

Sedimentation is the gravitational settling of particles having a density greater than water. There are four general classes of particle settling (Type I through IV) with the type of settling generally a function of the sediment concentration. Type I is discrete particle settling that occurs for concentrations generally ≤ 100 mg/L and is the most common assumption for settling calculations. Type II is flocculent settling and generally occurs at concentrations above 200 mg/L. Type II settling is dependent on clarifier or sedimentation depth and, as most sedimentation basins operate under Type II non-ideal settling, performance is depth-dependent. Type III is hindered or zone settling (500 to 1000 mg/L) and Type IV is compression settling (> 1000 mg/L). Sedimentation basins or detention basins receiving a highly concentrated particulate loading will operate under Type III and Type IV settling in the lower depths of the basin; however, these types of settling are usually not considered in the design of settling basins. All settling estimates require knowledge of particle properties such as particle size, density, and settling velocity.

Type I sedimentation is the most common assumption employed when considering first-order or initial settling calculations. The potential for particulate removal can be determined using a number of theories, for example numerical constitutive computational fluid dynamic models, Hazen's analytical model (Urbonas and Stahre, 1993), or surface overflow theory (Metcalf and Eddy, 2003), although use of numerical constitutive settling models is rare in practice (Cristina et al., 2002). While Hazen's model has advantages as compared to overflow (surface loading) rate theory and can account for non-ideal settling conditions, overflow rate design still remains the conventional practice in the United States for design of settling basins. Measurement or calculation of particle terminal settling velocity is required for a particle of given diameter and density. Additionally, the mass or number fraction of the total gradation for a given particle size increment must be known or determined.

Flotation is similar to gravitational sedimentation except in the opposite direction. Flotation processes (e.g., flocculation) that utilize the net buoyancy between a gas-solid floc are not commonly used for stormwater treatment. Typically floatable materials such as trash, debris, and hydrocarbons are removed through treatment processes that utilize the location of these pollutants on the water surface for removal.

Density separation, in particular sedimentation, is the most common unit process for stormwater treatment. Density separation is a very important unit operation in stormwater treatment as inorganic particulates typically account for the majority of the mass of pollutants in stormwater, and usually a significant fraction of the particulate matter is of sufficient size to be readily settable in the water column. Sedimentation is the primary removal mechanism in treatment facilities such as detention basins, wet ponds, and treatment wetlands, with these types of stormwater treatment facilities often incorporating a sedimentation forebay for removal of the larger particulates, with finer particulates settling in the primary water quality pool. Settling basins are well suited for treating storm flows where there is sufficient area for their placement. Unit processes for removal of floatable material generally are not limited due to space constraints and are appropriate for flows with appreciable amounts of trash, debris, or hydrocarbons.

Nearly all storm flows are amenable for treatment with density separation unit processes. The presence or absence of floatable material dictates whether baffles or inverted outlets are appropriate for treatment facility design. The consistent presence of sediments in storm flows makes treatment through sedimentation appropriate for most runoff. Sediment properties (e.g., size distribution, density) will influence the design of settling basins and the realistically achievable removal efficiency.

4.2.4 Volatilization

Volatilization is the process whereby liquids and solids vaporize and escape to the atmosphere. Compounds that readily evaporate at normal pressures and temperatures are volatile compounds. While these compounds are not frequently detected in highway runoff, volatile/ semi-volatile organic carbons (VOCs/ SVOCs) are sometimes present including various petroleum hydrocarbons (e.g., BTEX and PAHs), gasoline oxygenates (MTBE), herbicides, and pesticides. VOCs can also be formed during some microbial and phyto-chemical oxidation-reduction transformations of other potential pollutants in highway runoff. Volatile compounds are usually highly soluble in water and will easily migrate to groundwater during infiltration practices. Therefore if these compounds are present, it is recommended to remove them prior to infiltration.

Volatilization from a free water surface is a considered a three-step process: 1) escape from water surface, 2) diffusion through boundary layer, and 2) advection and hydrodynamic dispersion into the atmosphere. Figure 4-1 depicts the volatilization process from a free water surface.

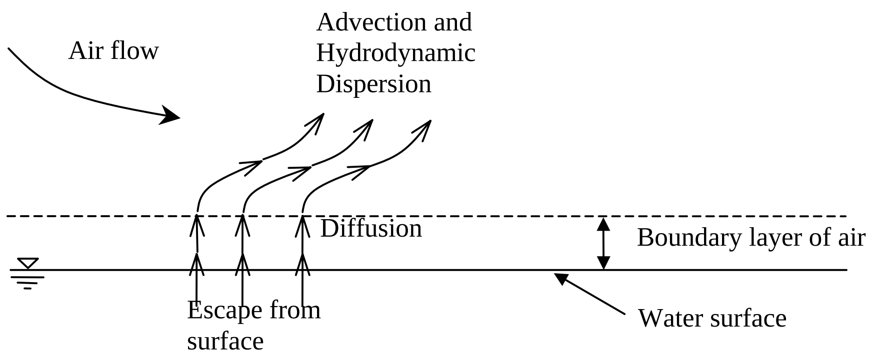


Figure 4-1. Physical unit process of volatilization from a free water surface.

Volatilization may also occur during evapotranspiration processes. Plants that readily take up volatile compounds may emit these compounds from their roots, stalks and shoots. This unit process should be considered a viable alternative if volatile or semi-volatile compounds have been detected or are expected to be present at the site. Sufficient surface area is likely needed for the types of BMPs that will promote volatilization.

Volatilization can often be integrated into a treatment process without the need for much (if any) additional planning or design of the treatment system. BMPs that include a large air-water interface, promote good air flow, are exposed to elevated temperatures or direct sunlight, or have significant aquatic or terrestrial vegetation may provide volatilization as a unit process. Volatilization can also be promoted through the use of aeration and evaporation enhancement controls.

4.2.5 Aeration

Aeration is the process of entraining air in the water column to increase dissolved oxygen levels. Urban water bodies are often impaired from low dissolved oxygen caused by high biochemical oxygen demand (BOD) or nutrient loads. Many types of beneficial microorganisms in aquatic treatment systems are more productive when dissolved oxygen is not limited. Also, some stormwater constituents will change form if aerobic conditions are not maintained.

Aeration should be considered if receiving waters are limited for dissolved oxygen concentrations or high BOD or nutrient loads or other oxygen-demanding substances, e.g., calcium-magnesium acetate (CMA), are expected from the site. For most stormwater treatment applications, it is desirable to have little to no energy requirements. Therefore passive aeration methods are often preferable. However, in some situations it may be necessary to have an energy source available to meet the aeration needs at a site.

Similar to volatilization and evaporation, aeration may occur by simply increasing the amount of open surface water since oxygen diffusion will occur naturally. The rate of diffusion, as well as the saturated dissolved-oxygen concentration, is dependent on the water temperature, salinity, and barometric pressure. Generally, as the temperature and salinity increase and the barometric pressure decreases, the dissolved oxygen diffusion rate decreases. If the microbial metabolism exceeds the oxygen diffusion rate, additional measures are necessary such as increasing the turbulence or mechanically introducing air to the water. Turbulence can be increased by promoting supercritical flows or by mechanical agitation. Other mechanical aeration methods include the use of submerged diffusers, air jets, and sprinklers. Turbulence can be increased most effectively if there is a significant drop in elevation where cascading water flow is possible. For any inlet or outlet structure that increases flow velocities, care must be taken to avoid scour and/or resuspension of particulates.

4.2.6 Natural Disinfection

Natural disinfection refers to the mitigation of stormwater borne pathogens through the use of non-chemical agents such as sunlight, ultraviolet light, and heat. Natural disinfection minimizes the danger and liability associated with the transportation and storage of chemicals that are used in chemical disinfection. Both natural and chemical agent disinfection lead to a partial destruction of pathogens and should not be confused with sterilization that leads to a complete destruction of all of the organisms (Metcalf and Eddy, 2003).

Ultraviolet (UV) disinfection is the primary natural unit process. Significantly less literature is devoted to heat and sunlight disinfection applications in stormwater as compared to ultraviolet disinfection. Ultraviolet light immobilizes stormwater-borne pathogens by penetrating pathogen cell walls and causing the formation of double bonds within the pathogens, thereby preventing replication and /or leading to the death of the organism (Metcalf and Eddy, 2003). High turbidity and TSS in the influent can reduce the effectiveness of UV disinfection (USEPA, 1999c). Pretreatment may be required to lower influent solids, adding to costs. UV disinfection may be more expensive than other disinfection alternatives; however, advances in technology and increases in the number of UV facilities continue to lower costs (USEPA, 1999c).

As previously mentioned, heat and ultraviolet light may both be used to immobilize pathogens. The sun is an abundant source of both heat and ultraviolet light. However, due to the paucity of data pertaining to the effectiveness of solar disinfection as well as reduced solar intensity during storms, solar disinfection is only recommended in conjunction with other unit processes. For example shallow detention ponds allow heating and the penetration of sunlight to the facility bottom thereby creating a less friendly environment for pathogens.

UV disinfection is generally suited to applications where the influent to the physical disinfection system will have low solids content. Solids provide places for pathogens to hide and also prevent the penetration of light. Natural disinfection should be chosen over chemical disinfection (specifically chlorine injection) if the influent could potentially have a high organic content. Chlorine reacts with organics reducing the available chlorine concentration for disinfection and can result in the formation of trihalomethanes (e.g., chloroform) and other chlorinated hydrocarbons. Trihalomethanes are the primary concern as these compounds are suspected carcinogens. For facilities that may need to discharge into sensitive receiving waters, natural disinfection may be more suitable since physical disinfection agents (e.g., UV, heat) leave no residuals in the effluent.

Projects that have identified pathogens as constituents may want to select either chemical or natural disinfection. Chemical agent disinfection is currently relatively cheaper; however, this may change in the future with improvements in technology and increased restrictions on the discharge of disinfection byproducts (e.g., trihalomethanes).

4.3 Biological Treatment Processes

Biological processes use living organisms (plants, algae, and microbes) to transform or remove organic and inorganic pollutants. Relevant processes for stormwater treatment have been divided into two broad categories: microbially-mediated transformations and uptake and storage.

4.3.1 Microbially-Mediated Transformations

These are the unit processes of microbial activity that promote or catalyze redox reactions and transformations. These processes include the degradation of organic pollutants, as well as the oxidation or reduction of inorganic pollutants. Microbially mediated transformations are chemical transformations primarily by bacteria, algae, and fungi that exist in the water column, soil, root zone of plants, and on wetted surfaces, such as leaves (Kadlec and Knight, 1996; Karthikeyan and Kulakow, 2003; Minton, 2005). Most microbes are concentrated in the upper layers (0.3 m) of soil and in the plant root zone. All microbially-meditated transformations are similar in concept; microbes facilitating transformations of constituents. However, fundamental differences in these transformations processes allow them to be separated into four (4) categories: metabolism; organic material decomposition and mineralization; inorganic transformations; and degradation of xenobiotic compounds. Each of these is discussed below as well as the applicability of microbially-mediated transformations to stormwater treatment. Further information on each of the four categories, including examples, may be found in Section 2.3.5 of the *Final Research Report*.

4.3.1.1 Metabolism

When microbes metabolize organic compounts, transformations occur as a result of respiration, which is a redox reaction. Redox reactions are chemical transformations involving the transfer of protons and electrons. Respiration is the process that releases the energy and nutrients from food sources so that they can be assimilated by organisms. Metabolic transformations that occur in stormwater treatment systems are largely influenced by the oxidation-reduction (redox) potential of the system.

4.3.1.2 Organic Material Decomposition and Mineralization

When microbes aerobically oxidize simple organic compounds, the process releases, or mineralizes, organically bound elements. Mineralization refers to the release of elements from organic matter to produce inorganic (mineral) forms. Most of the inorganic elements released by mineralization are in forms more available as nutrients to higher plants and microbes. Once released through mineralization, the elements can be further transformed by specific microbes, sequestered by binding to other inorganic constituents, or by sorbing to non-degradable organic matter (humus). Mineralization is an important source of nitrogen, sulfur, phosphorus, and other nutrients for plants and microbes.

4.3.1.3 Inorganic Transformations

Some microbes can enzymatically oxidize or reduce metals during respiration, affecting metal solubility and reactivity. Inorganic transformations are used to treat metals in the practice of bioremediation. Example reactions include the oxidation of ferrous (Fe^{2+}) to ferric (Fe^{3+}) ions precipitating ferric hydroxides or phosphates, the reduction of sulfate to sulfide causes forming insoluble metal sulfides, the nitrogen cycle and others.

Of all transformation processes, conversions of nitrogen species (e.g., ammonia, nitrate) as part of the nitrogen cycle are probably the most significant in stormwater treatment systems.The nitrogen cycle includes nitrogen transformations facilitated by microbes (primarily bacteria) in addition to uptake and release of nitrogen from multi-cellular organisms and abiological processes. The microbial transformations of ammonification, nitrification, denitrification, and fixation are of interest for improvements in runoff water quality. For a more detailed discussion of the nitrogen cycle, see Section 2.3.5.1.4 of the *Research Report*.

4.3.1.4 Degradation of Xenobiotic Compounds

In addition to simple organics, various microbes (primarily heterotrophic bacteria) are able to use more complex organics (such as xenobiotic compounds, compounds foreign to biological system) as energy sources during metabolism, often resulting in microbial decomposition to less toxic compounds. In some cases, xenobiotic compounds undergo incomplete degradation, and the products may be as or more toxic than the parent compound. For example, trichloroethene (TCE) is degraded to vinyl chloride rather quickly. However, subsequent degradation of vinyl chloride, a carcinogen, usually occurs slowly.

Degradation can occur aerobically or anaerobically, although both processes occur relatively slowly (thus requiring long residence times). Significant degradation is possible for phenols, phthalate esters, naphthalenes, chlorinated benzenes, and nitroaromatics in aerobic conditions. Some compounds degrade more rapidly in anaerobic conditions, including carbon tetrachloride, chloroform, lindane, phenol, and methylene chloride (Minton, 2005). Complete degradation of some constituents may require alternating aerobic and anaerobic conditions (Knapp and Bromley-Challenor, 2002). Under the right conditions, some microbes can transform xenobiotic compounds even when the chemical is not the primary energy source (cometabolism). Cometabolism is important for the breakdown of chlorinated solvents, polychlorinated biphenyls, and many PAHs, which is the basis for bioremediation of organic pollutants.

4.3.1.5 Applicability to Stormwater Treatment

Stormwater treatment that incorporates vegetation and or permanent water bodies usually has a diverse microbial population, and it is not possible to optimize conditions for all beneficial species. However, basic habitat requirements for all microbes include a substrate to colonize (e.g., soil, plant roots, leaf surfaces), appropriate nutrients including carbon sources (and absence of toxics), and sufficient moisture. Other factors affecting microbial populations include pH, vegetation density (including type of plants), organic amendments, and presence of oxygen or other electron acceptor.

Microbially-mediated transformations can remove dissolved nitrogen species (e.g., nitrate), metals, and simple and complex organic compounds. Many transformations only occur in the presence of specific microbes. Soils may be inoculated with desirable microbes to promote specific reactions, or if the initial microbial population is low. Transformations occur relatively slowly and require long residence times. Temperature affects microbial growth and transformation rates. Generally, increasing the temperature increases transformation kinetics.

Stormwater BMPs (wetlands, swales, retention ponds) that facilitate these processes can require relatively large land areas and therefore may not be suitable in highly urbanized areas. They may also have limited applications in arid climates, areas with long dry seasons, and cold climates. Some microbially mediated processes have the potential for stream warming and should not be used where effluents discharge to temperature sensitive water bodies, such as cold water habitats. Nitrification may result in leaching of nitrate from the system, which is of particular concern in areas with water quality impairment due to nutrient-enrichment.

4.3.2 Uptake and Storage

Uptake and storage refer to the removal of organic and inorganic constituents by plants and microbes through nutrient uptake and bioaccumulation. Nutrient uptake converts required micro- and macro-nutrients into living tissue, whereas bioaccumulation incorporates compounds (e.g., pollutants) into an organism, regardless or in excess of what is immediately needed. Uptake and storage processes are generally not major pollutant removal processes in stormwater treatment systems due to the extended retention times required for such processes. Characteristics of essential nutrients may be found in Table 2.2 in the Final Research Report. A more complete discussion factors affecting uptake and storage is included in Section 2.3.5.2 of the *Final Research Report*.

Plants and microbes require essential nutrients to sustain growth, which may be assimilated from the water column or from soil solution through metabolic processes. The specific forms in which nutrients exist are determined largely by pH and redox potential and affect the assimilation of these nutrients. Removal of phosphorus is the most significant uptake mechanism in stormwater treatment systems.

In addition to nutrients, various algae, and wetland and terrestrial plants accumulate organic and inorganic constituents in excess of their immediate needs (bioaccumulation). Bioaccumulation is an evolutionary response to scarcity in the natural environment, and is the basis of phytoremediation. Bioaccumulation processes contribute to the effectiveness of constructed wetlands for wastewater treatment. The ability to remove chlorinated solvents, petroleum hydrocarbons, herbicides, insecticides, and phenolic compounds has been investigated for wetland and terrestrial plants.

4.3.2.1 Uptake of organics

Plant uptake of organics is a function to the organic compound’s solubility, hydrophobicity (octanol-water distribution coefficient K_{ow}), and polarity. Moderately hydrophobic polar compounds may be sorbed by the roots and translocated, while more hydrophobic neutral compounds will be sorbed by roots, but not translocated (USEPA, 2000a). Soil conditions, plant physiology, and evapotranspiration rates also affect the uptake of organic compounds. Appropriated plant species should be selected based on the compound(s) of interest.

4.3.2.2 Uptake of Metals

Uptake and storage can be used to remove dissolved metals, nutrients (phosphorus and nitrogen), and organic compounds. The process is suitable where soil properties and water quality are adequate to support organism growth.

Uptake of metals depends on metal bioavailability. Low bioavailability may explain why there are so few hyperaccumulators of lead, as lead tends to form insoluble precipitates. Organic matter excreted by roots can increase metal bioavailability by lowering the pH, or by forming metal chelates. As a general rule, readily bioavailable metals for plant uptake include cadmium, nickel, zinc, arsenic, selenium, and copper. Moderately bioavailable metals are cobalt, manganese, and iron. Lead, chromium, and uranium are not very bioavailable. Lead can be made much more bioavailable by the addition of chelating agents to soils.

The ability of plants to accumulate and store metals varies greatly. The term hyperaccumulator applies to plants that can accumulate metals at concentrations 100-fold greater than concentrations found in the tissue of non-hyperaccumulators. Metal tolerance is the primary characteristic of these plants. These plants are also capable of translocating the metal from the root to plant stems and leaves. Hyperaccumulating plants have affinities for specific metals, and metal affinity may vary within different species within the same genus. Consequently, significant metal uptake by plants will not occur unless the appropriate species are selected. Number of hyperaccumulating plant species for specific metals can be seen in Table 2.3 of the *Research Report*.

The efficiency of uptake processes varies by season, latitude, and plant species and may be reduced in cold or arid climates. Other factors effecting uptake include soil characteristics, presence of nutrients, suitability of plant species, and presence of symbiotic microbes. Many systems require appreciable land. Some uptake and storage processes have the potential for stream warming and should not be used where effluents discharge to temperature sensitive water bodies, such as cold water habitats. Concentrations in stormwater treatment systems may not be high enough for processes such as metal hyperaccumulation or organic compound reduction to occur.

4.4 Chemical Processes

The chemical characteristics of stormwater, e.g. pH, alkalinity, hardness, redox conditions, organic carbon, and ionic concentrations, affect the partitioning and speciation of stormwater pollutants, which in turn dictates the type of UOPs necessary to treat those pollutants. Three common chemical UOPs applied in the field of stormwater treatment include sorption, coagulation/flocculation, and chemical agent disinfection.

4.4.1 Sorption

While often inseparable from filtration unit operations, sorption refers to the individual unit processes of both absorption and adsorption. Absorption is a physical process whereby a substance of one state is incorporated into another substance of a different state (e.g., liquids being absorbed by a solid or gases being absorbed by water). Adsorption is the physiochemical adherence or bonding of ions and molecules (ion exchange) onto the surface of another molecule. In stormwater treatment application, particularly for highway runoff, the primary pollutant types targeted with absorption unit processes are petroleum hydrocarbons, while adsorption processes typically target dissolved metals, nutrients, and organic toxicants such as pesticides and polycyclic aromatic hydrocarbons (PAHs). Different types of filter media may provide either or both of these unit processes.

Sorptive unit processes are a spectrum of specific mechanisms that range from surface complexation to precipitation and such processes are generally designed for solute mass transfer onto high surface area materials, generally engineered media. In stormwater, solutes with contemporary interest include phosphorus and metals. In combination with these sorptive processes, unit operations such as filtration can be an effective treatment control for dissolved and particulate-bound metal and phosphorus species. Mass transfer of dissolved species can occur to either engineered media or through mass transfer to stormwater runoff particles (partitioning) and then be separated as particulate-bound constituents through filtration. These processes are a dependent on a variety of factors and are potentially reversible. A discussion of mass transfer mechanisms is included in Section 2.3.6.1 of the *Research Report*.

Engineered media such as oxide coated filter media with high surface area and amphoteric (pH-dependent) surface charge can be utilized to carry out the combined unit operations of filtration and processes of surface complexation for a range of treatment configurations for in-situ, decentralized treatment or centralized stormwater runoff treatment. Considerations for use of engineered media and examples of applications are included in Section 2.3.6.1 of the *Research Report*

Equilibrium isotherms are an important tool to describe the equilibrium between aqueous and solid (media) phases for a known combination and concentration of solute(s), media, water chemistry, media/solution ratio, experimental geometry and hydrodynamics. Isotherms indicate the adsorption capacity of a media or particulate solid under a prescribed or given set of conditions. An extensive discussion of sorbtion isotherms, including theory, applications and limitations is included in Section 2.3.6.1 of the *Research Report*. Surface complexation (SC) theory, which is an alternative method of modeling sorption, is also discussed in this section of the *Research Report*.

4.4.2 Precipitation/Coagulation/Flocculation

Precipitation, coagulation, and flocculation are actually three processes that occur simultaneously or in quick succession. Precipitation is the process by which a pollutant is transformed from primarily a dissolved state to a solid state. Coagulation is the process by which colloidal particles are destabilized so that particle growth can occur. Flocculation is the process by which fine particles collide to form larger particles that can be readily removed through filtration and settling. While these three processes will occur naturally, the addition of chemicals is usually necessary to accelerate the process.

Engineered chemical and physical flocculation is beginning to be applied in specific applications for stormwater runoff. Depending on parameters such as mixing, pH, ionic strength, and particle properties natural flocculation can begin within several hours to 12 hours of initial runoff. Natural flocculation, while generally not accounted for, can have a significant impact on stormwater runoff clarification in unit operations such as sedimentation basins or in detention/retention facilities.

When existing treatment technologies do not provide enough treatment to achieve water quality goals, the use of chemicals may be necessary. The types of pollutants typically targeted with precipitation/coagulation/flocculation processes include fine and colloidal particulates, dissolved metals, and phosphorus. The disadvantage of using these processes in stormwater treatment applications is the generation of potentially significant quantities of sludge that must be properly handled and disposed. Depending on the particular chemicals used, the effluent may not be suitable for discharge because of reduced or elevated pH, high dissolved aluminum or iron concentrations, or presence of other undesirable by-products.

The conditions and factors that enhance precipitation and flocculation processes are highly dependent on the chemicals being used; the primary factors include pH, temperature, and hardness. Other factors such as the particle-size distribution, free ion concentration, and the electro-negativity of colloidal particles will also influence these processes.

4.4.3 Chemical Disinfection

Chemical disinfection refers to the mitigation of stormwater-borne pathogens through the use of chemical agents such as chlorine and its compounds, and ozone. Chemical disinfection immobilizes pathogens through a variety of mechanisms including damage to pathogen cell walls, alteration of pathogen cell wall permeability, alteration of the colloidal nature of the protoplasm of the pathogen, alteration of the DNA or the RNA of the pathogen, and the inhibition of pathogen enzyme activity (Metcalf and Eddy, 2003). Chemical disinfection is used more extensively in wastewater applications with a wider range of chemical agents (see Section 2.3.6.3 of the *Research Report* for complete list). Since wastewater technologies are often adopted for stormwater applications the list of agents currently used for stormwater disinfection could expand in the future. A discussion of factors affecting disinfection performance is included in the same section of the *Research Report*.

Projects that have identified pathogens as constituents of concern must select either chemical or natural disinfection. These are the only two unit processes discussed in this document that specifically target pathogens. Chemical agent disinfection may be relatively cheaper than natural disinfection and hence may be a more suitable choice for a tight budget. Chlorine disinfection leaves a residual in the effluent that may provide added benefits by preventing the re-growth of pathogens and improving downstream water quality. Projects that anticipate the use of upstream BMPs that significantly reduce organic content and suspended solid content will result in an increase in the efficiency of any chemical facilities downstream. Potential impacts of selected constituents on the use of chorine and ozone are included in Tables 2.4 and 2.5 of the *Final Research Report*.

CHAPTER 5 SELECTION OF DISTRIBUTED BMPS AND BMP TREATMENT SYSTEMS

Several of the individual unit processes discussed above complement each other or work in unison and cannot be easily analyzed separately in actual treatment facilities. For example, detention basins may reduce the total runoff volume due to infiltration and evapotranspiration (ET), as well as attenuate peak flows, which cause particulates to settle out. Furthermore, some BMPs can be modified to include unit processes that are typically not accommodated for in their design, such as including infiltration in the design of a swale (e.g., dry swale). Since several BMPs may include multiple unit processes, the placement or order of BMPs and BMP components within a treatment system should be carefully considered to maximize the effectiveness of the design. The recommended approach herein is to use the concept of the treatment train based on the following general progression:

1. Minimize flow rates and/or volume of runoff from the urbanized drainage area (hydrological control and LID).
2. Remove bulk solids (pretreatment: > 5mm)
3. Remove settleable solids and liquid floatables (coarse primary treatment: >75 µm; fine primary treatment: >10 µm)
4. Remove suspended and colloidal solids (secondary treatment: > 0.1-25 µm)
5. Remove colloidal, dissolved, volatile, and pathogenic constituents (tertiary treatment)

It is important to note that some stormwater BMPs, such as vegetated swales, may be used as either primary and/or secondary components of a treatment train. Furthermore, tertiary treatment may be provided in BMPs that provide secondary treatment, such as constructed wetlands. Therefore, it may be more useful to categorize BMPs (and their components) according to the unit treatment processes that they provide. Table 5-1 maps the unit processes discussed in Chapter 4 to the BMPs that typically accommodate the process.

Table 5-1. Structural BMPs categorized by fundamental unit processes.

Fundamental Process Category (FPC)	Unit Operation or Process (UOP) Target Pollutants	Common BMPs
Hydrologic Operations	Flow and Volume Attenuation	Extended detention basins Retention/detention ponds Wetlands Tanks/vaults Equalization basins
	Volume Reduction *All pollutant loads*	Infiltration/exfiltration trenches and basins Permeable or porous pavement Bioretention cells Dry swales Dry well Extended detention basins
Physical Treatment Operations	Particle Size Alteration *Coarse sediment*	Comminutors (not common for stormwater) Mixers (not common for stormwater)
	Physical Sorption *Nutrients, metals, petroleum compounds*	Engineered media, granular activated carbon, and sand/gravel (at a lower capacity)
	Size Separation and Exclusion (screening and filtration) *Coarse sediment, trash, debris*	Screens/bars/trash racks Biofilters Permeable or porous pavement Infiltration/exfiltration trenches and basins Manufactured bioretention systems Engineered media/granular/sand/compost filters Hydrodynamic separators Catch basin inserts (i.e., surficial filters)
	Density, Gravity, Inertial Separation (grit separation, sedimentation , flotation and skimming, and clarification) *Sediment, trash, debris, oil and grease*	Extended detention basins Retention/detention ponds Wetlands Settling basins, Tanks/vaults Swales with check dams Oil-water separators Hydrodynamic separators
	Aeration and Volatilization *Oxygen demand, PAHs, VOCs*	Sprinklers Aerators Mixers (not common for stormwater)
	Natural Disinfection *Pathogens*	Shallow detention ponds Ultra-violet systems
Biological Processes	Microbially Mediated Transformation (can include oxidation, reduction, or facultative processes) *Metals, nutrients, organic pollutants*	Wetlands Bioretention systems Biofilters (and engineered bio-media filters) Retention ponds Media/sand/compost filters
	Uptake and Storage *Metals, nutrients, organic pollutants*	Wetlands/wetland channels Bioretention systems Biofilters Retention ponds
Chemical Processes	Chemical Sorption Processes *Metals, nutrients, organic pollutants*	Subsurface wetlands Engineered media/sand/compost filters Infiltration/exfiltration trenches and basins
	Coagulation/Flocculation *Fine sediment, nutrients*	Detention/retention ponds Coagulant/flocculant injection systems
	Ion Exchange *Metals, nutrients*	Engineered media, zeolites, peats, surface complexation media
	Chemical Disinfection *Pathogens*	Custom devices for mixing chlorine or aerating with ozone Advanced treatment systems

5.1 Highway LID Techniques

LID techniques refer to the hydrologic source control and practices that apply to runoff prior to entering the primary ("trunk" line) drainage system. LID is extremely applicable to the urban highway environment because it allows for BMPs to be constructed in small footprint areas and in variable topographic conditions. Most LID facilities will be located as close as possible to the source of runoff (i.e., roadsides). Some of these facilities are easily incorporated into the existing highway environment, while others due to safety or other concerns, may be more difficult to implement. Design provisions must be provided to ensure that there will be adequate drainage that does not affect or impede vehicle passage in the case of clogging or overflow of the facility. Strategic locations of LID facilities can also be used to address non-point source pollution from the right-of-way by controlling runoff at the source. The following subsections describe a few of the common LID techniques that are applicable to the highway environment. For information on additional LID techniques, as well as specific design guidance, refer to the *LID Design Manual* developed as part of this project.

5.1.1 Permeable Pavement

Permeable/porous pavement capitalizes on infiltration to minimize surface runoff. There are two general types of permeable pavement, monolithic and interlocking. Monolithic permeable pavement is constructed by leaving out most of the fine fraction of aggregate of the pavement mix, and placed like regular pavement (Urbonas and Stahre, 1993). For low traffic volume applications (e.g., parking lots), the permeable layer is generally placed on top of a thick granular porous layer that promotes infiltration directly beneath the pavement surface. However, for highway situations a structural layer must underlay the permeable layer to support heavier truck loads. Frequently this structural layer is impermeable, forcing stormwater to drain to the roadside to be treated, infiltrated, and/or routed to the storm drain system. The Oregon Department of Transportation has good success placing thick layers (2 to 4 inches) of open-graded asphalt mixes over conventional dense-graded mixes on high volume routes. Depending on traffic levels these mixes last 10 to 15 years.

Open-graded asphalt friction courses (OGFCs), also known as “popcorn” or porous asphalt, have been in use in the United States for a number of years. These mixtures are typically placed in thickness of approximately ¾ inch. The mixes reduce tire spray and road noise and improve traction. These friction courses must be placed on a sound structural layer. Over the past 30 years, there have been several refinements to the traditional OGFCs, such as using polymer-modified asphalt binders to improve durability and using larger maximum aggregate sizes to increase void ratios.

For low-speed areas, such highway rest stops, an alternative to monolithic permeable pavement may be used. Interlocking permeable pavement, also known as pavers, is made up of numerous interlocking pavement stones that can have seams and/or holes in them to allow easy drainage of water. The holes and seams in the interlocking pavers may be filled with sand, gravel or grass to assist high levels of infiltration (Huber et al., 2005). The design of permeable pavement must recognize the elevation of groundwater and bedrock with respect to the pavement. If the groundwater and/or bedrock are located too close to the porous pavement the infiltration ability of the pavement could be drastically hindered (Urbonas and Stahre, 1993). Groundwater contamination issues must also be taken into account and include groundwater uses, risks due to industrial activities in the catchment, use and traffic levels on the porous pavement, and use of de-icing salts on the street (Pitt et al., 1996). Before the pavers are laid the native soil is properly compacted and a geotextile fabric is placed followed by a thick layer of uniform graded rock base. The fabric serves to prevent fines in the native soil from contaminating the base rock. Above the rock base, a granular graded filter material is placed. The base material would now be considered ready for the placement of the pavers.

Porous pavement maintenance and longevity are nearly always a concern and generally should be constructed in low traffic areas. Oregon's success with thicker sections of open-graded layers is the exception. Monolithic porous pavement generally clogs within a 1 to 2 -year time span unless high levels of maintenance are performed, including vacuuming and pressure washing. Oregon does not perform any maintenance on their open-graded mixes probably due to the high operating speeds and significant rainfall where these materials are used. Interlocking porous pavement does not generally have the clogging issue, depending on the paver hole size (Field et al., 2000). These maintenance practices can be very costly to ensure porous conditions of the pavement. In regions with severe winters, there have been few problems associated with porous pavement and the freeze thaw cycle (Pitt and Voorhees, 2000). Figure 5-1 shows two types of permeable pavements installed in parking lots.

Figure 5-1. Permeable Pavement: Interlocking Blocks (left) and Porous Asphalt (right).

5.1.2 Roadside Infiltration/Exfiltration Trenches

Roadside infiltration and exfiltration trenches are gravel or sand filled trenches that receive runoff either directly from the road surface or after a pretreatment BMP, such as a sedimentation basin, to remove large particles and reduce the maintenance burden. The difference between an infiltration trench and an exfiltration trench is that an infiltration trench *infiltrates* runoff from the trench into the underlying soil, while an exfiltration trench *exfiltrates* runoff from the trench into a perforated underdrain that is connected to the storm drain system. As such, infiltration trenches are much more sensitive to the native soil type and structure than exfiltration trenches. In both types of facilities, engineered media, such as oxide-coated sand (OCS) may be used to increase the adsorption of heavy metals and other highway pollutants. In a design called a partial exfiltration trench (PET) both infiltration and exfiltration occurs (Sansalone and Buchberger, 1995). Figure 5-2 is an example of an infiltration trench used to treat runoff from the adjacent roadway.

Figure 5-2. Example of an Infiltration Trench Used to Treat Roadway Runoff.
Source: California BMP Handbooks (www.cabmphandbooks.com)

5.1.3 Roadside Swales and Bioswales

Roadside swales are vegetated roadside conveyances that provide the functionality and benefits of vegetated treatment. Bioswales, as defined herein, are modified swales that use bioretention media and selected vegetation to enhance the effect that swales have on water quality, runoff volume, and peak runoff rate. These enhanced BMPs perform the same functions as traditional grassed swales by serving as a conveyance structure and filtering and infiltrating runoff (see Section 5.3.6 for a discussion on vegetated swales). They differ, however, because the use of bioretention media enhances infiltration, water retention, and nutrient and pollutant removal. Like bioretention cells, bioswales encourage infiltration in order to retain runoff volume and use a variety of physical, chemical, and biological processes to reduce runoff pollutant loadings. Refer to Section 5.4.1 for information on the design characteristics, mechanisms, and maintenance considerations for bioretention systems. Additional guidance is included in the *LID Design Manual* developed as part of this project. Figure 5-3 shows two examples of roadside swales. If additional treatment or volume losses are desired, swales such as these could be easily retrofit with bioretention media.

Figure 5-3. Example Roadside Bioswales.

5.1.4 Curb Cuts or Perforated Curbs

For retrofit situations or when curbs are required by standard design specifications to stabilize the roadway edge or for safety considerations, curb cuts may be a feasible alternative for reducing stormwater runoff from roads, highways, and parking lots. This can be accomplished by maintaining flow patterns to vegetated areas, rather than collect the runoff into inlets along the curb and convey it to a centralized location in pipe systems. The curb cuts can be designed as openings or as inlet structures with drop structures and open backs to dissipate the energy. Vegetated filter strips, bioretention areas, or gravel sumps can be added at the discharge points for infiltration of runoff and filtration of pollutants. Level spreaders, such as stone or concrete weirs can be used to dissipate the energy and spread the flows at the discharge points. Figure 5-4 is an example curb cut leading to a roadside bioretention cell. Bioretention is discussed in detail below in Section 5.4.1 and in Section 5.0 of the *LID Design Manual*.

Maintenance of these systems will typically require removal of sediment and gross solids that settle at the curb opening. The systems should be inspected regularly for blockage, especially if there is potential for ponding in the travel lanes.

Figure 5-4. Example of Curb Cuts to a Bioretention Cell (left) and Swale (right).

5.1.5 Filter Strips and Bioslopes

Filter strips are sloped, grassed or vegetated shoulders that receive sheet flow from adjacent impervious surfaces. Bioslopes are filter strips that are enhanced with bioretention soils and often include a level spreading trench to promote interflow through the soil matrix. The cut or fill slope is compacted to the standard high density requirements for unpaved, roadside areas with the exception of the top 12 to 18 inches. The remaining area is filled with a bioretention soils mixture (e.g., compost, sand, mulch, etc.). Gross solids, trash, and sediment are captured in the surface soils and vegetated layers. A geotextile may be used to stabilize the interface with the compacted subgrade. At the bottom of the slope a swale or infiltration trench is typically constructed for water quality, volume, or flow rate control. Figure 5-5 is an example of roadside filter strip that could be retrofit as a bioslope if additional treatment or volume losses are desired.

Figure 5-5. An Example of a Roadside Filter Strip.
Source: Caltrans 2003.

5.1.6 Narrow Pavement Designs

Narrower road widths and alternative road profiles can reduce stormwater runoff volumes and mitigate its impacts. However, the horizontal and vertical geometry for high volume roadways are generally dictated by recommendations found in AASHTO policies on geometric design (AASHTO, 2004). Reducing design width along highway travel ways or shoulders is generally not recommended due to safety considerations, but in some cases a narrower design may actually reduce the speed of traffic. Extensive guidance on narrowing roadways and reducing traffic speeds for neighborhood streets is provided by the Institute of Transportation Engineers (ITE, 2005). This guidance can be used for rest stops access roads or other highway pullout areas.

5.1.7 Near Roadway LID Opportunities

For the purposes of identifying LID opportunities, roadway sections may be evaluated according to the geometric classification. As previously stated in Section 3.1.5, the classes under this classification scheme include: at-grade, cut, fill, or bench sections. Each class of roadway provides unique opportunities for the incorporation of LID design elements for both new construction and retrofit applications. While these opportunities are often site specific and may be subject to additional design constraints depending on existing infrastructure and the crown of the roadway (e.g., super elevated), the general concepts are valid. The LID zones presented in Table 5-2 represent parts of the roadway that offer similar opportunities for the incorporation of LID techniques.

Table 5-2. LID zone descriptions and recommended stormwater treatment elements.

LID Zone	Zone Description	Recommended LID Techniques
Zone 1	Areas beyond roadway clearing limits	Bioretention areas, pocket wetlands, revegetation, conventional or innovative BMPs
Zone 2	Cut or fill slope face	Curb cuts, filter strips, bioslopes, erosion control
Zone 3	Roadside ditch, depression, or other drainage feature	Swales, infiltration/exfiltration trenches, bioretention area
Zone 4	Travel way surface	Permeable pavement, narrow pavement designs

5.1.7.1 At-Grade Sections

This class refers to sections of roadway that are constructed on relatively flat surfaces as opposed to the other section types that are constructed on slopes. At-grade sections involve minimal amounts of earthwork as compared to the other sections and the constructed road surface is approximately at the same elevation as the surrounding areas. Since hydraulic potential is limited for these geometric roadway configurations, the ability to route runoff much further than the limits of clearing may be challenging. If traffic speeds are low at a particular site, such as for pull-out areas or rest stops, permeable pavement or narrow pavement designs may be appropriate depending on state DOT standard specifications (Zone 4). For higher speed limit roadways, the LID design should focus on zones 2 and 3 that include the use of curb cuts, bioslopes, swales, infiltration/exfiltration trenches, or bioretention areas. For very flat areas and depending on the infiltrative capacity of the native soils, perforated underdrains may be necessary to route runoff down gradient and away from the roadway. More specific design guidance for these types of LID practices can be found in the *LID Design Manual*.

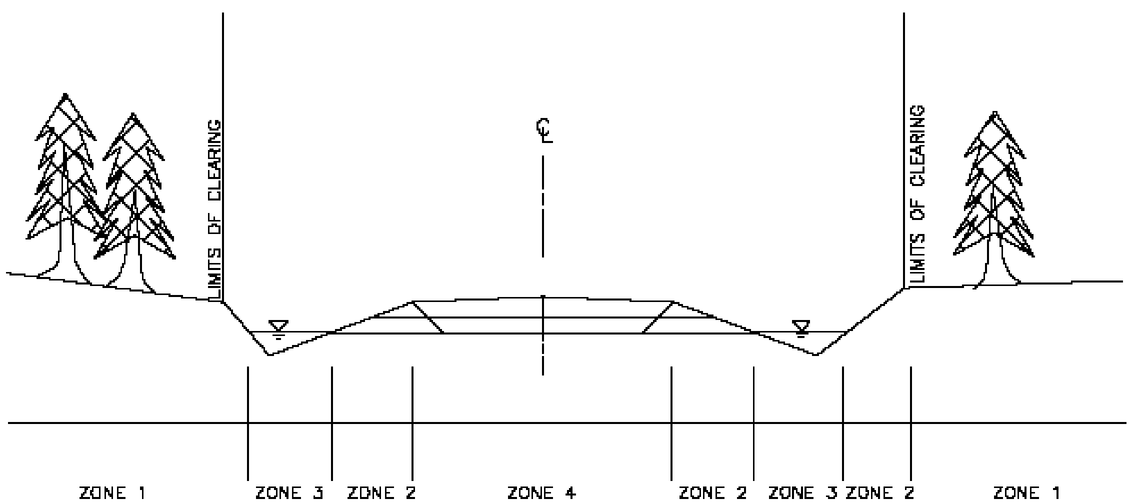


Figure 5-6. Roadway BMP and LID Zones for At-Grade Sections.

5.1.7.2 Cut Sections

Cut sections occur where a roadway is built by cutting through ground that is at a much higher elevation than the finished road surface. The traveled roadway is bordered on both sides by cut slopes. For cut sections, roadway runoff is unable to sheet completely off to one side since both sides of the road are at higher elevations than the road surface. However, sheet flow from the top of the slopes may find its way down to the roadway. Roadside ditches and storm drains are typically used to drain cut sections. The ability to utilize areas outside of the limits of clearing is not really possible. Therefore, as with at-grade sections, LID practices for cut sections should focus on zones 2 and 3 including curb cuts, bioslopes, infiltration/exfiltration trenches, and swales.

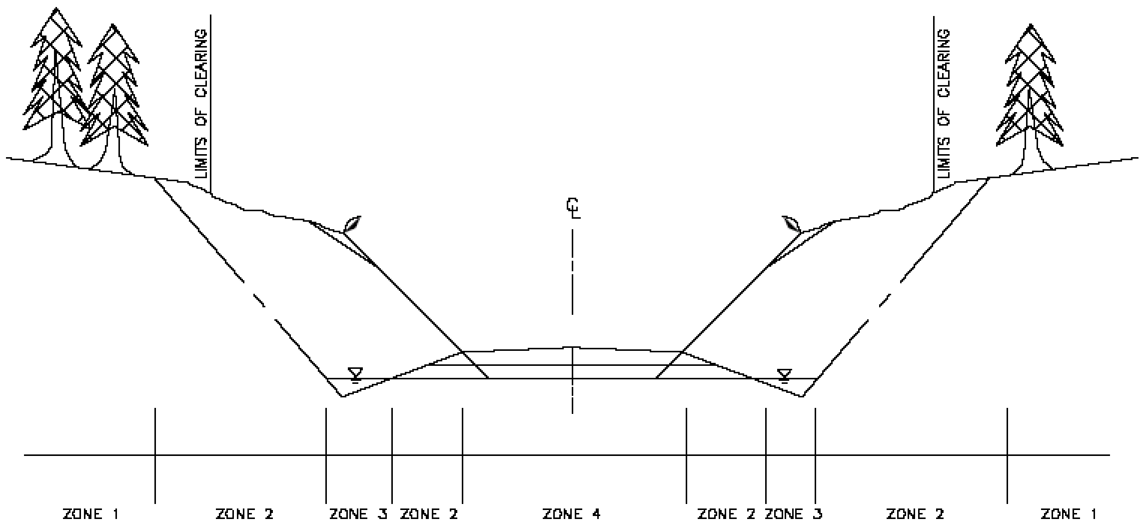


Figure 5-7. Roadway BMP and LID Zones for Cut Sections.

5.1.7.3 Fill Sections

Fill sections refer to the parts of a roadway that are built on top of artificial embankments that make the finished road surface elevation much higher than the surrounding ground elevations. Since the road surface is higher than the surrounding areas, sheet flow off the side of the roadway can occur on both sides (unless super elevated). The road surface may be able to drain whether roadside ditches and other concentrated flow paths are present or not. In the absence of roadside ditches, an LID zone 3 does not exist. Therefore, LID practices are limited to those appropriate to the other three zones including curb cuts, infiltration/exfiltration trenches, and bioslopes. Permeable pavements and narrow pavement design may be appropriate for non-highway situations. The use of soil amendments to maximize infiltration and pollutant removal may be more cost effective for fill sections than other configurations because fill material is needed anyway to raise the elevation of the roadway.

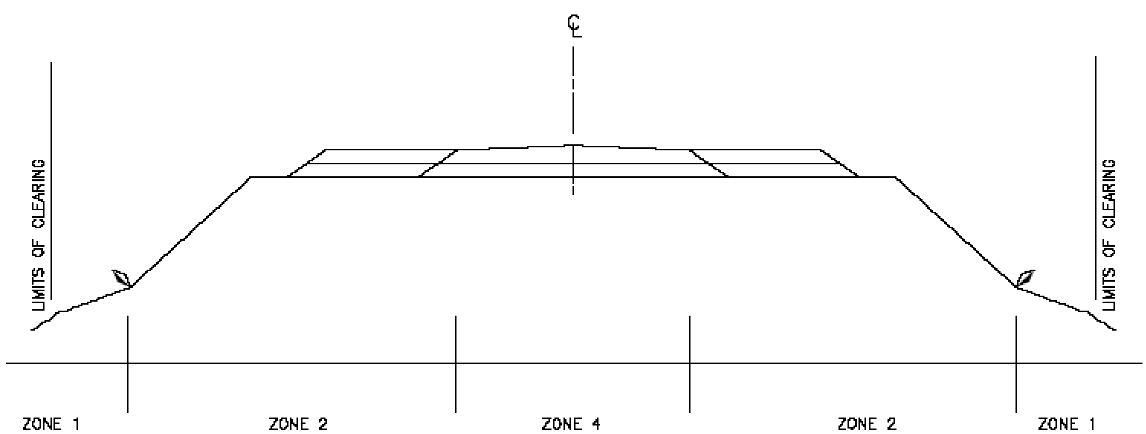


Figure 5-8. Roadway BMP and LID Zones for Fill Sections.

5.1.7.4 Bench Sections

Bench sections refer to combination cut and fill sections where one half of the road way is a cut section and the other half of the roadway is a fill section. This approach is useful for balancing cut and fill in road development projects, which is desirable because the cost of hauling material to or from the project site can significantly increase the overall project costs. Bench sections can usually be drained using methods that are applicable for both fill sections and cut sections. Concentrated flow path drainage elements, such as swales and culverts, and sheet flow elements, such as bioslopes, can be combined to provide custom solutions for bench sections.

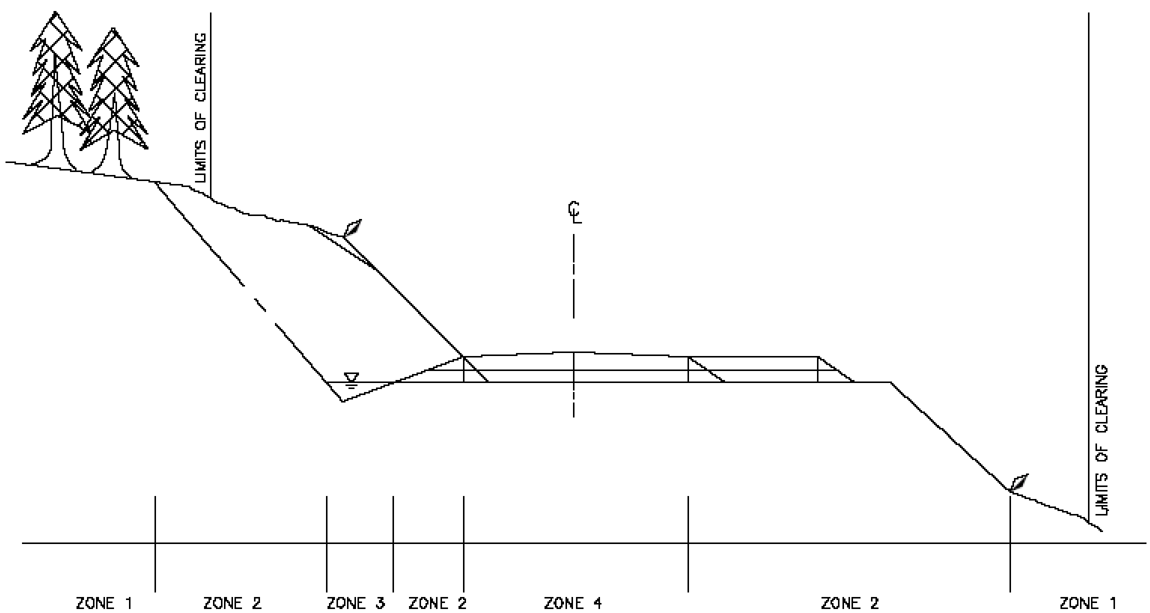


Figure 5-9. Roadway BMP and LID Zones for Bench Sections.

5.2 Pre-Treatment Devices

Pre-treatment devices are normally intended to target bulk and floatable pollutants such as trash, debris, and oil and grease. These devices typically do not remove bulk solids smaller than 5 mm and only target free oil floating on the water surface rather than emulsified oil. There are a large number of these devices, many of which are proprietary and many that are simple public works-type devices. These devices can be generally classified as inlet devices, in-line devices, and floating traps.

The initial treatment provided by pre-treatment devices can reduce the potential for clogging or obstruction of downstream components, enhance the performance of primary treatment BMPs, and reduce costs by reducing the required maintenance frequency of downstream facilities. Pre-treatment devices should be considered for all treatment systems, particularly in the highway environment where litter is prevalent.

Pre-treatment devices are relatively inexpensive compared to other BMPs, but are limited in the constituents that they can effectively treat. Pre-treatment devices tend to be durable and usually require little maintenance of the structures making them a cost-effective component of a treatment train.

5.2.1 Inlet Devices

Inlet devices include screens, baskets, grates, and bar racks placed at or near storm drain entry points designed to prevent trash and debris from entering into the storm drain system. These may include simple grates over drop inlets, screens placed at the curb side, or hanging or drop-in baskets within the catch basin. Screens and racks are low profile BMPs often installed in the drainage path of stormwater conduits and therefore require little to no additional area. Catch basins may also contain absorbent booms, media filters, and filter fabric to remove free oil and smaller particles, but it is recommended that large trash and debris be removed prior to the use of these types of inserts. Inlet devices are typically used as pre-treatment devices to remove large items from stormwater prior to further treatment by other downstream treatment facilities and are sometimes used to protect pumps and pipe inlets from being impacted by stormwater-borne solids. Overly restricted flow paths can lead to clogging and flooding; therefore frequent cleaning is required to minimize headloss, and avoid clogging.

5.2.2 In-Line Devices

Pre-treatment in-line devices are structural BMPs that are applied beyond the drain inlet; targeting mostly gross solids and floatables. Examples of pre-treatment in-line devices include screens, baffles, water quality manholes, and end-of-pipe devices such as release nets and trash traps. In-line devices can be placed in various locations in the treatment train, and therefore may target a variety of constituents. Release nets and trash traps may require space at the end of the conduit where they are usually installed for the storage of debris prior; however, space requirements for in-line devices are generally low. In-line devices include many public works devices and an endless list of proprietary devices typically used as pre-treatment for more effective BMPs.

Table 5-3. Principal Unit Processes for In-line Treatment

In-line Device	Principal Unit Process for Pollutant Removal
Screens	Size Separation
Baffles	Skimming/Density Separation
Water Quality Manholes	Skimming/Density Separation
Release Nets	Size Separation
Trash Racks	Size Separation

In-line devices are most suitable for removal of gross pollutants, and floatable materials such as debris and free oil and grease. Parking lots, roads or other areas that are likely to be a source of hydrocarbons should be considered for an in-line treatment device with oil absorbent material. Most urban land uses are good candidates for an in-line device that will effectively remove floatable debris and gross pollutants. These considerations are also pertinent to floatable traps discussed in the following section.

5.2.3 Floatables Traps

Floating debris traps and booms provide removal of floatable solids from stormwater. These devices depend on the unit process of flotation which causes floatables to rise to the surface where traps and booms are usually placed. Inlet booms can be placed at curb inlets to capture coarse sediment and absorb oil and grease, while floating booms and skimmers can be placed on open water surfaces for the removal of floatable gross solids, floatable liquids, and particles. Floating booms and passive skimmers are generally easy to use, have a low capital cost, and are suitable for retrofit situations. Floating skimmers and booms do not store volumes or pass flows, but inlet absorbents may impede flows and cause ponding upstream of the inlet. The materials trapped by booms and skimmers, depending on the design and the amount of captured debris, can alter the hydraulics of the influent stream. Most simple skimmers and booms float on the open water surface or are placed in front of or within a storm drain inlet so generally there are not additional area requirements. Costs for booms and skimmers are relatively low compared to primary treatment BMPs. Maintenance costs for the devices themselves are typically low with the majority of operation and monitoring costs spent for regular monitoring, removal, and disposal of captured pollutants.

5.3 Primary Treatment BMPs

Primary treatment generally targets settleable solids that are larger than about 100 microns. A discussion of the associated unit processes and the factors that enhance or hinder these processes for each of the primary treatment BMPs is provided below.

5.3.1 Tanks and Vaults

Tanks and vaults are typically underground structures that are used to store and either slowly release or later reuse surface water. For the purposes of this document, tank is used as a generic term for referring to plastic or corrugated metal storage tanks, as well as oversized, low-gradient pipes (detention pipes) or similar structures. Vaults will be used to refer primarily to reinforced concrete or similar structures used for the same purposes as tanks.

Storage facilities, such as tanks and vaults can potentially provide peak attenuation, and sedimentation. Depending on the particular application, storage facilities can also provide storage and reuse of stormwater for landscaping or other uses. Figure 5-10 provides two examples of underground storage tanks that can be used for temporary storage of stormwater runoff.

Figure 5-10. Example of Concrete Vault (left) and Modular Storage Tank (right). Source: ARC (2001) and Atlantis (2005).

5.3.1.1 Associated Unit Processes and Potential Design Enhancements

Tanks and vaults are primarily sedimentation and peak attenuation devices; as such they will typically only remove coarse materials and suspended solids and the pollutants associated with them. The performance of tanks and vaults depends on the design of the storage areas, the design of the inlet and outlet structures, and the quality and flow characteristics of the influent. Regular removal of trapped materials from tanks and vaults will ensure consistent performance and decrease negative impacts to downstream treatment system components (TSCs). Further treatment of the effluent from tanks and vaults basins must be provided in most cases.

While tanks and vaults are well suited for space-limited applications, other storage-type BMPs such as detention ponds are often more desirable for stormwater quality treatment. Tanks only provide a limited level of water quality treatment due to the absence of vegetation and soils/media, which can provide additional unit processes such as vegetative filtration, biological uptake, and sorption. To increase the number of unit processes provided by these simple structures, some innovative enhancements include placing filtration or oil-absorbent media within the vaults or incorporating baffles and inverted outlets into the design. In fact, there are several proprietary vault-based devices available that include these design enhancements. These devices are discussed in detail in a later section of this document.

5.3.1.2 Recommended Pretreatment

Tank and vaults typically require little pretreatment since these devices themselves can be used as pretreatment for other BMPs. However, tanks and vaults may benefit from the installation of screens and racks as a cheap way of keeping larger pollutants out. The addition of a sedimentation forebay is recommended whenever possible. The forebay can be created by simply adding a baffle to isolate a section of the tank or vault. The forebay size should be about 25 percent of the total volume. The inclusion of a forebay facilitates maintenance and may reduce the frequency of maintenance of the main chamber.

5.3.1.3 Hydraulic Considerations

Tanks and vaults are relatively simple to design in terms of hydraulics. The inlet and the outlet structures should be selected and sized to provide the required flow properties. The tanks or the vaults themselves must be sized to store the entire water quality design volume and the outlet structure must be sized to provide the desired hydraulic retention time. Inlet structures must be designed to safely bypass flows that exceed the design volume in offline configurations.

5.3.1.4 Maintenance Considerations

Tanks and vaults must be regularly inspected to determine if cleaning is needed, to ensure that vents, inlet structures and outlet structures are unobstructed and also to ensure that cracks that may cause water to enter or leave the tank are identified and fixed. Regular removal of accumulated material is needed to ensure consistent performance.

5.3.1.5 Required Surface and Subsurface Area

The only surface area required by sub-surface installations is space for maintenance access, which is generally via a manhole. Vaults are typically placed beneath parking lots or road surfaces for easy access to the manhole cover. The subsurface area required is proportional to the total volume of the tank or vault and the desired detention depth. Above ground installations may take up a significant area which is again proportional to the design volume. Area requirements can be easily estimated by calculating the bottom or the top area of the structure in question, or by consulting the manufacturer of the device.

5.3.1.6 Cost Considerations

Concrete vaults are expensive and hence should be limited to small drainage areas while corrugated metal pipe tanks are relatively inexpensive. A preliminary cost estimate (FHWA, 1996) for concrete vaults can be obtained from the equation below:

$$C = 38.1(V / 0.02832)^{0.6816} \qquad [5\text{-}1]$$

Where:

C = construction cost estimate (1995 dollars) and
V = volume of storage (cubic meters) for the maximum design event frequency.

Both concrete vaults and corrugated metal pipe vaults are durable treatment system components.

5.3.1.7 Safety and Aesthetics

Storage tanks and vaults are typically installed below ground and hence do not present a significant public safety hazard or impact the aesthetics of the installation site. For above-ground installations, storage tanks and vaults can be painted and strategically placed to enhance the overall aesthetics of an area. Above ground installations must incorporate all the necessary signage for public safety. Also all vaults and tanks must have lids that should remain closed during normal operation to ensure public safety. For large tanks where authorized individuals may need to periodically enter during maintenance, a confined space entry program must be developed and explicitly followed. If the access to the site is near a the roadway, appropriate safety measures (e.g., lane closures, cones, warning signs, flaggers, etc.) must be taken according to the specific safety requirements of the state DOT. If standing water is likely to occur greater than 72 hours, a mosquito control program should be developed.

5.3.2 Oil Water Separators

Oil/water or oil/grit separators are designed to remove gross pollutants including petroleum hydrocarbons, grease, sand, and grit. Interception of solid particles through settling, and flotation of oils and other floatables are processes occurring within an oil/water separator. The conventional oil/water separator consists of a large chamber divided by baffles into three sections. The first chamber acts as an equalization chamber where grit and larger solids settle and turbulent flow slows before entering the main separation chamber. After entering the main chamber, solids settle to the bottom and oil rises to the top, according to Stokes' law. Larger oil/water separators contain a sludge scraper, which continually removes the captured settled solids into a sludge pit. The oil is also removed by an oil skimmer operating on the water surface. At the end of the separation chamber, all oil particles having a diameter of larger than the critical size have theoretically risen to the surface and have been removed by an oil skimmer. Small units usually do not contain an oil skimmer, sludge scrapper, or sludge pit. While they are less costly due to the absence of moving parts, they require more frequent cleaning and maintenance. These smaller units have been shown to be as effective as the larger more expensive units, if they receive proper maintenance at regular intervals.

There are two common designs for oil-water separators: the American Petroleum Institute (API) separator and the Coalecing Plate Separator (CPS) (Figure 5-11). The API separator consists of three chambers divided by baffles and first chamber acts as an equalization chamber where grit and larger solids settle and turbulent flow slows before entering the main separation chamber. The CPS, which is generally smaller than the API, uses a single baffle and a series of oil-attracting coalecing plates in the main chamber.

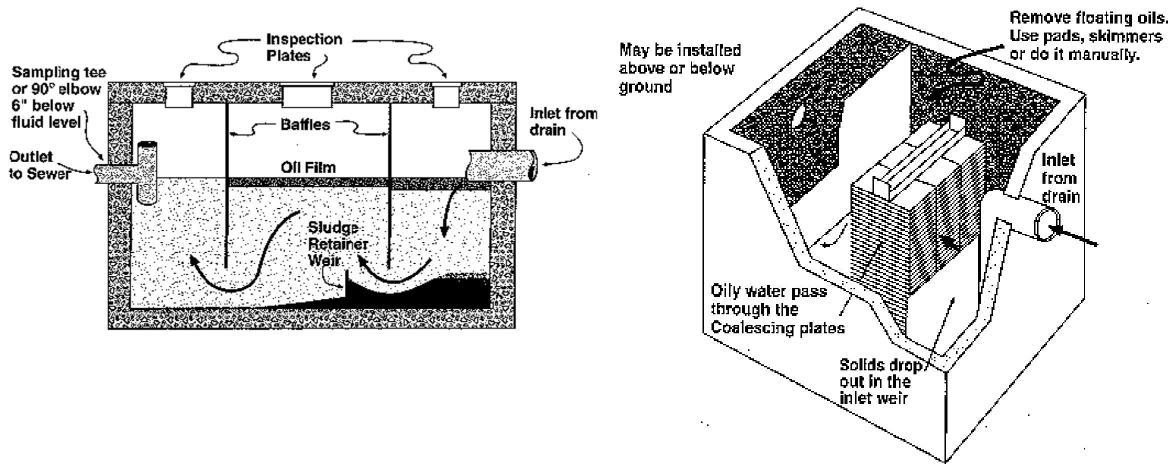


Figure 5-11. Common Oil-Water Separator Designs.
American Petroleum Institute (API) Oil/Water Separator (left) and Coalescing Plate Separator (CPS) (right).
Source: King County (2005), http://Dnr.Metrokc.Gov/Wlr/Indwaste/Oilfact.htm

5.3.2.1 Associated Unit Processes and Enhancement Considerations

Gravity separation that relies on the density differences between oil and water is the most basic type of unit process used. Oil will rise to the water surface unless some other contributing factor such as a solvent or detergent interferes with the process. For gravity units, this density difference is the only mechanism by which separation occurs. Other technologies, such as dissolved air flotation, coalescing plates, and impingement coalescing filters, enhance the separation process by mechanical means. The addition of coalescing unit to the oil/grit separator can dramatically increase its effectiveness as well as greatly reduce the size of

5.3.2.2 Pretreatment Considerations

The use of a catch basin or an interceptor tank as a pretreatment device would keep the coarse material from entering the oil/water separation tank.

5.3.2.3 Hydraulic Considerations

The oil/water separator design process is based on the Stokes law and includes the following steps:

1. Determine the droplet rise velocity (V_T) of the critical droplet size using Stokes' Law:

$$V_T = \left(\frac{g}{18\mu}\right)\times(\rho_w - \rho_o)\times d^2 \qquad [5\text{-}2]$$

Where: V_T = rising velocity (terminal velocity) of oil droplets (ft/s)
g = acceleration due to gravity (ft/s^2)
μ = absolute viscosity of water (lbm/ft·s)
ρ_w = density of water (lbm/ft^3)
ρ_o = density of oil (lbm/ft^3)
d = droplet diameter (ft)

2. Once the critical rise-rate (V_T) and maximum flow (Q) have been determined, the effective horizontal separation area is calculated from the equation:

$$A_H = \frac{Q}{V_T} \qquad [5\text{-}3]$$

Where: A_H = Area (ft^2)
Q = flow rate (ft^3/s)
V_T = rising velocity (terminal velocity) of oil droplets (ft/s)

This formula, also known as Hazen's principle, is commonly used in oil/water separator design. Often, large areas are required for effective separation. However, stacked coalescing plates can be used to create the necessary separator area in a limited space. The efficiency of a separator also depends upon the flow rate: as the flow increases, the separator performance decreases. Therefore, a separator must be designed to accommodate the maximum expected flow for a given rainfall event.

Selecting the critical (or design) density of oil is another relevant factor in the design of an oil/water separator. The heaviest oil presumed to be present is used in determining the critical rise velocity. In general, specific gravities of oils range from about 0.82 to 0.95. The separator will be most efficient for the lowest oil densities. Water temperature also affects oil/water separator performance. At lower temperatures, separation becomes more difficult, and therefore, the lowest temperature routinely encountered should be used in the design. Ambient ground temperatures a few meters below the surface can be used to estimate water temperatures for an underground installation. Also, ambient air temperatures during cooler weather can be used. The solids content of the wastewater must also be considered for separator design. After the basic dimensions of the separator have been calculated, sufficient volume within the separator must be added for solids storage between cleanings.

5.3.2.4 Maintenance Considerations

Problems with oil/water separators can be attributed largely to poor maintenance by allowing waste materials to accumulate in the system to levels that hinder performance and to levels that can be readily scoured during intermittent high flows. When excess oil accumulates, it will be forced around the oil retention baffle and make its way into the discharge stream. Also, sludge buildup is a major reason for failure. As waste builds up, the volume in the chamber above the sludge layer is reduced and therefore the retention time is also reduced, allowing oil to be discharged. Therefore, the efficiency of oil/water separators in trapping and retaining solids and hydrocarbons depends largely upon how they are maintained. They must be designed for ease of maintenance and be frequently maintained. Apparently, few oil/water separators built for stormwater control are adequately maintained.

Ease of maintenance must be considered when designing separators, including providing easy access. Maintenance on these devices is accomplished by using suction equipment, such as a truck mounted vacuum utilized by personnel trained to handle potentially hazardous waste. The vacuum is used to skim off the top oil layer and the device is then drained. In larger devices, the corrugated plates are left in place, but otherwise, they are lifted out along with any other filter devices that are present. The sludge is then vacuumed out or shoveled out and any remaining solids are loosened by spraying hot water at normal pressure.

Maintenance of parallel plate units and coalescing filters is similar. The separator is drained and the plates are washed by spraying. If there is inadequate space, then the plates will need to be lifted from the separator for effective cleaning. Cleaning should occur when coating of the plates is evident and before accumulations begin to clog the spaces. Cleaning of polypropylene coalescing tubes is also accomplished by lifting out the tube bundles and cleaning with a hose or high pressure water spray to remove accumulated oil and grit. Sludge is removed from underneath the coalescer supports and the coalescers are then replaced.

5.3.2.5 Cost Considerations

The construction costs for oil-water separators will vary greatly depending on their size and depth. The construction costs (provided in 1993 dollars in the reference) for cast-in-place oil-water separators range from $5,000 to $16,000, with the average oil-water separators costing around $8,500 (Schueler, 1992). For the basic design and construction, the pre-manufactured units are generally less expensive than those that are cast in place (Berg, 1991). Maintenance costs will also vary greatly depending on the size of the drainage area, the amount of the residuals collected, and the clean out and disposal methods available (Schueler, 1992). The cost of residuals removal, analysis, and disposal can be a major maintenance expense, particularly if the residuals are toxic and are not suitable for disposal in a conventional landfill.

5.3.2.6 Safety and Aesthetics

As with tanks and vaults, oil-water separators are typically installed as sub-surface controls and therefore do not significantly impact public safety or aesthetics. Like other controls, frequent maintenance prevents aesthetic impacts from large quantities of accumulated material and the associated odors. Well designed and maintained oil water separators are expected to have very minimal aesthetic impacts. Typically, these devices do not require confined space entry, but if needed a confined space entry program would need to be developed. If the access to the site is near a the roadway, appropriate safety measures (e.g., lane closures, cones, warning signs, flaggers, etc.) must be taken according to the specific safety requirements of the state DOT. If standing water is likely to be present greater than 72 hours, a mosquito control program should be developed.

5.3.3 Hydrodynamic Devices

Hydrodynamic devices are flow-through devices with a settling or separation unit to remove sediments and other pollutants by means of swirl action or indirect filtration. They remove settleable solids and floatables such as oil and are not effective in removing fine sediments and dissolved constituents including metals and nutrients. Normally they are not designed for flow attenuation, such as precast settling vaults (StormVault™, Stormceptor®), or filtration. However vendors do provide supplemental features that would attenuate flow and would remove dissolved pollutants by filtration.

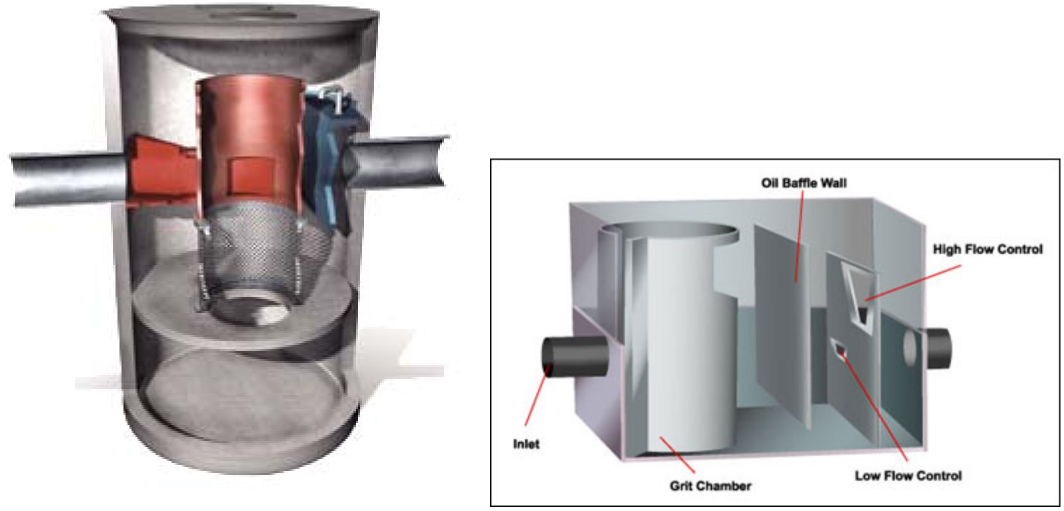


Figure 5-12. Example Hydrodynamic Separation Devices.
CDS Technologies (2005) (Left) and Vortechnics (2005) (Right).

5.3.3.1 Pollutant Removal Processes

The primary unit processes provided by hydrodynamic devices include density and size separation. They provide enhanced gravitational separation of floatable and settleable pollutants from stormwater. The enhanced separation is accomplished by causing the influent to flow in a swirling motion in a circular chamber. The swirling action creates velocity differentials that encourage pollutants to move toward the center of the vortex, while relatively clean water from the outer fringes of the vortex exits the system. Baffles are often included in the design of these devices to facilitate the removal of floatable solids and oil and grease.

5.3.3.2 Performance Evaluation

While there are numerous proprietary hydrodynamic designs with a range of reported effectiveness values, all such devices can be assumed to be effective for separation of gross pollutants such as floatable material, trash and debris, and sediment sizes greater than about 75 μm. The appeal of hydrodynamic devices lies in their ability to provide solids removal at high flow rates. Targeted pollutants for hydrodynamic devices include primarily solids and floatables. Sorbent booms and other enhancements are needed for hydrodynamic devices to be able to mitigate dissolved constituents.

Table 5-4 provides median effluent concentrations, for illustrative purposes only, from 13 monitoring studies of hydrodynamic devices from the ASCE/EPA International BMP Database (2003). One study was excluded as the drainage area was primarily open space.

Table 5-4. Median Effluent Values for Hydrodynamic Devices

Water Quality Constituent & Units	Median Value
Solids, Total Suspended (mg/L)	57
Phosphorus, Total (mg/L as P)	0.14
Nitrate + Nitrite, Total (mg/L as N)	0.25
Nitrogen, Kjeldahl, Total (mg/L as N)	0.8
Ammonia (mg/L as N)	0.25
Nitrate Nitrogen, Total (mg/L as N)	0.94
Copper, Total (ug/L as Cu)	12
Lead, Total (ug/L as Pb)	6
Zinc, Total (ug/L as Zn)	70

These devices are either operated as stand-alone single units or in combination with upstream or downstream treatment system components. However, the removal of large trash and debris prior to treatment in hydrodynamic devices is generally recommended.

5.3.3.3 Sizing Considerations

Hydrodynamic devices are flow-through facilities and are therefore sized to pass specified flows. Most proprietary devices will include various combinations of weirs, screens, orifices and baffles, which act as flow control structures. Sizing and selection is accomplished by estimating the flow through the weir, orifice or baffle that acts as the flow control structure for the treatment chamber. Another important design consideration is the maximum flow capacity of the structure. If the device is used in an on-line configuration, a by-pass mechanism will typically be provided in the form of a weir to pass high flows. Off-line structures do not have to be designed to pass high flows and may not need extra flow controls to by-pass high flows. Based on the target flow rate required to be captured and treated, manufacturers typically provide software or tables to facilitate design and selection of structures.

5.3.3.4 Maintenance Considerations

Manufacturers of proprietary hydrodynamic devices recommend frequent inspections. The frequency of inspections depend more on site conditions than on unit size. Areas with unstable soils or heavy winter sanding may require more frequent maintenance (Vortechnics, 2004). Vacuum trucks are usually the most effective method and convenient method of cleaning hydrodynamic devices. Absorbent pads are recommended for removal of oil and hydrocarbons that accumulate since disposal of absorbent pads is usually cheaper than the disposal of oil water emulsions. Trash and debris can be removed with nets if it is desirable to separate the trash for the other pollutants (Vortechnics, 2004). Some of the hydrodynamic devices incorporate removable baskets that can be removed and emptied. Recommended maintenance frequencies range for once a year to four times a year. For consistent results, follow the manufacturer's recommendations.

5.3.3.5 Required Surface and Subsurface Area

The hydrodynamic devices currently available are mostly subsurface devices. The only surface area need for these devices is access for maintenance. The sub-surface space requirements for any of the hydrodynamic devices can be obtained from the manufacture once design flow rates are known. Hydrodynamic devices are designed for space limited applications, therefore the space requirements for these devices range from very low to low.

5.3.3.6 Cost Considerations

Currently, most of the available hydrodynamic devices are proprietary devices such as Continuous Deflection Separation Units, the Stormceptor®, the Downstream Defender™ and Vortechs™ units. The cost of the systems are typically specified per cfs of flow treated and cost range from $10,000 to $62,000 for precast units capable of handling 2 cfs to 26 cfs (USEPA, 1999d).

5.3.3.7 Safety and Aesthetics

Hydrodynamic devices are typically installed below ground and hence do not impact the aesthetics of an area. In the unlikely event that an above ground installation is required, the hydrodynamic device can be painted and strategically placed to enhance the overall aesthetics of an area. If the access to the site is near a the roadway, appropriate safety measures (e.g., lane closures, cones, warning signs, flaggers, etc.) must be taken according to the specific safety requirements of the state DOT. Public access to the devices should be restricted by using heavy or locking manhole covers. If standing water is likely to be present greater than 72 hours, a mosquito control program should be developed.

5.3.4 Sedimentation Ponds and Forebays

Sedimentation ponds and forebays mainly provide density separation of stormwater constituents. Sedimentation ponds and forebays are mainly used to remove settleable solids prior to treatment by other TSCs. Like most detention-type BMPs sedimentation ponds and forebays can function both as density separation and peak attenuation BMPs depending on the design of the outlet and inlet structures used. Sedimentation tanks and forebays protect other TSCs downstream, and decrease the maintenance frequency requirements of downstream TSCs by capturing sediment that would otherwise negatively impact potentially more sensitive and expensive treatment system components downstream. Figure 5-13 is an illustration of a sedimentation forebay within a detention basin.

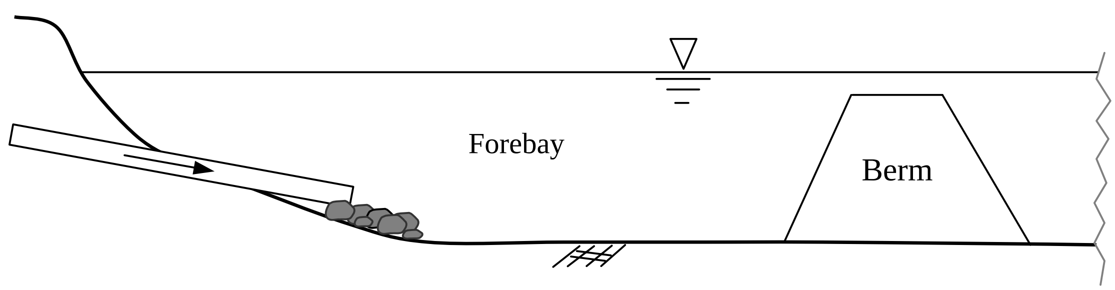


Figure 5-13. Sedimentation forebay near a detention basin inlet.

5.3.4.1 Associated Unit Processes and Potential Design Enhancements

As the name implies, sedimentation basins depend for treatment primarily on the density characteristics of coarse solids. Therefore, designs that maximize the hydraulic residence time of the captured volume are preferable.

The performance of a pre-settling basin or a sedimentation basin/forebay depends on the design of the basin itself, the design of the inlet and outlet structures, and the quality and flow characteristics of the influent. Since this class of BMPs is meant to trap coarse sediments, an indicator of their performance is how quickly downstream TSCs silt up. Regular removal of trapped materials from pre-settling basins will ensure consistent performance and decrease negative impacts to downstream TSCs. Sedimentation ponds/basins/forebays will typically only remove coarse materials and their associated pollutants. Further treatment of the effluent from this class of BMPs generally must be provided, unless coarse sediment is the only target constituent.

5.3.4.2 Recommended Pretreatment

Unless there is a constricted inlet structure, pre-settling basins generally do not need pretreatment. However a rack or screen at the inlet can serve as a cheap means of excluding large objects that could potentially disturb and re-suspend previously captured sediment or obstruct inlet/outlet structures

5.3.4.3 Hydraulic Considerations

For detention pond applications the sedimentation forebays are sized to provide 15% to 30% of the total volume of the pond. Check with the local regulatory agency for exact sizing requirements. Over sizing a sedimentation basin/forebay may lead to added cost with little benefit and under sizing the sedimentation basin/forebay component of a pond system may cause siltation of downstream components, increased maintenance requirements and lessen pollutant removal abilities of downstream systems. The outlet structures of pre-settling basins must be designed to prevent trapped sediment from being flushed out into downstream TSCs, and the flow rates out of the pre-settling basin must be such that the pre-settling basin does not become a bottleneck in the system (IDEQ, 2001).

5.3.4.4 Maintenance Considerations

Regular removal of trapped materials from pre-settling sedimentation basins/forebays is required to maintain performance. Maintenance frequencies will depend on site specific conditions such as sediment loads and basin capacity. The berms or impoundments of the basin must be regularly inspected since breaches may cause property damage. Inlet and outlet structures must be routinely inspected and also inspected after large storms to remove any obstructions that may negatively impact performance.

5.3.4.5 Required Surface and Subsurface Area

Sedimentation basins/forebays are typically surface TSCs and therefore occupy a fair amount of space. For detention-type applications, the width of the basin typically matches the width of the rest of the pond. The length of the basin is then selected to provide the volume equivalent to the selected percentage of the total volume of the pond system. The surface area required by the pre-settling basin is easily calculated in this case. For flow-through applications the size of the pre-settling basin required is less obvious and depends on the flow rates to be passed, the quality of the influent and other factors.

5.3.4.6 Cost Considerations

The cost of sedimentation basins/forebays basins is comparable to the cost of ponds of similar sizes. The maintenance frequencies of sedimentation basins/forebays can, however, be much higher than the maintenance frequency of ponds. The benefits of properly designed sedimentation basins/forebays far outweigh the costs since pre-settling basins increase the longevity and effectiveness of the entire system while facilitating maintenance.

5.3.4.7 Safety and Aesthetics

Similar to ponds, sedimentation basins and forebays can be constructed to be aesthetically pleasing and safe. Landscaping and regular maintenance can improve and maintain the aesthetics of pre-settling basins while preventing the development of bad odors and the incidence of vectors. However, a mosquito control program may need to be developed if standing water occurs for more than 72 hours. Public safety can be ensured through the use of fencing or mild side slopes.

5.3.5 Surface Filters (Filter Fabrics)

In stormwater applications, the unit process of particle size separation can be accomplished through the use of media filters or surface filters. Media filters provide filtration by directing the influent through layers of media such as sand, peat, zeolite, etc. Surface filters on the other hand consist of a membrane or a fabric that serves as a barrier to stormwater borne particulates that are larger than the pore openings in the filter. Media filters are much more commonly applied in stormwater applications because they are robust, easy to construct and also because a variety of filter media with diverse properties are readily available. Most media filters use filter fabrics to prevent filter media from clogging other drainage components such as pipes and outlet structures. For example filter fabrics are used as pipe wraps in infiltration and media filtration facilities such as sand filters and infiltration trenches to prevent intrusion of the filter media into the outlet pipe system. In addition to being used as components of media filtration systems, filter fabrics can be used as standalone filters. Standalone applications of filter fabrics are not very prevalent in stormwater management. However, the use of filter fabrics for erosion and sediment control, particularly as construction site BMPs, is very prevalent. There are also several catch basin insert and curb inlet designs that utilize geotextiles for coarse sediment filtration.

In general filter fabrics require less space than media filters and are easier to maintain. Filter fabrics, however, require a frame to provided external structural support since fabrics tend to have adequate tensile strength and negligible compressive strength. Filter fabrics are made from various materials with a wide array of drainage properties. Commonly used filter fabrics can be categorized based on manufacturing methods as either non-woven filter fabrics or woven filter fabrics.

Figure 5-14. Example surface filter application in roadside trench drain.

Source: http://www.geotextile.com/drainage/drainage.htm

5.3.5.1 Applications

Filter fabric is used for various stormwater applications including the following:

- To keep granular media out of perforated pipe systems in media filtration system
- To provide drainage paths behind sheet piles, retaining walls and other subsurface barriers
- To prevent clogging of outlet pipe systems in detention and retention ponds
- To provide filtration in catch basin insert systems
- As turbidity curtains and silt screens to provide containment of turbid regions in water bodies
- To retain soils while allowing water to pass.
- For a host of construction site erosion control and stream bank stabilization practices.

Advantages of filter fabrics include: low cost compared to granular media, ease of maintenance, reduced excavation, faster and simplified construction, greater flexibility in selecting structurally sound backfill options and biodegradable options.

Figure 5-15. Example application of a filter fabric used as a silt fence.

5.3.5.2 Associated Unit Processes and Potential Design Enhancements

The primary unit process associated with surface filters are physical unit processes such as filtration and screening. Surface filters are typically much thinner than media filters, therefore the contact time between the filter and the influent is much shorter, limiting occurrence of chemical unit processes.

The single most important factor influencing the efficiency of the filtration process is percent open area. The performance of filter fabrics is related to the size of the pore openings in the fabric. If the filter is well maintained and its integrity is not compromised, the filter will at a minimum provide good removals of target particle sizes that are larger than the filter pore openings.

Please note that filter fabrics will only provide removal of solid particles and not dissolved constituents. Only particulate bound bacteria, nutrients, metals, and other particulate bound constituents may be removed through the filtration process under the right conditions. Factors that impact the filtration efficiency in surface filters include:

- The effective filter area
- Size of filter fabric apertures
- Percent open area of filter fabric
- Pressure drop across the filter
- Resistance of filter to fluid flow
- Swelling effect of influent on the filter
- Compressibility of the filter fabric under fluid pressure
- Size of suspended particles
- Tendency of particles to flocculate
- Rate of formation of filter cake
- Resistance of filter cake to fluid flow

Once filtration has begun, the ability of a filter to keep filtered solids out will depend on the duration of the filtration operation and the size, shape and nature of the suspended solids. A more detailed discussion of filtration process is provided in Section 4.2.2.

5.3.5.3 Recommended Pretreatment

Surface filters may benefit from the use of coarse screens and trash rack as pretreatment controls. Screens and racks prevent sharp objects or large objects from causing structural damage to the filter. Floatables can potentially expedite filter clogging; therefore, practices that prevent coarse solids and floatables from reaching the filter surface constitute good pretreatment. For erosion control and applications filters may be used without pretreatment to protect other drainage structures. In soil stabilization and media filtration applications the soil or the media serves as pretreatment by precluding large or sharp objects and floatables from reaching the filter surface.

5.3.5.4 Hydraulic Considerations

Filters should ideally operate under streamline flow. Poiseille's law modified by Kozeny and Carmen can be used to model liquid flow through porous media. The rate of flow through a filter is directly proportional to the pressure drop across the filter and the area of the filter perpendicular to the direction of flow, but inversely proportional to the viscosity of the effluent and the thickness of the filter in the direction of the flow. Filter fabrics are often used to keep soils out of drainage system components while allowing the passage of water. For such applications, Darcy's equation can be used to model fluid flow through soils:

$$Q = KiA \quad [5\text{-}4]$$

Where: Q = flow rate (ft^3/s)
K = hydraulic conductivity (ft/s)
i = hydraulic gradient of the soil (ft/ft)
A = cross sectional area soil (ft^2)

Equation 5-5 presents an expression that can be used as a criterion for determining acceptable water passage through a soil geotextile/filter system.

$$K_g > i_s K_s \quad [5\text{-}5]$$

Where: K_g = hydraulic conductivity of the geotextile (ft/s)

i_s = Hydraulic gradient of the soil (ft/ft)
K_s = hydraulic conductivity of the soil (ft/s)

In summary the performance of filter fabrics depend on the surrounding conditions and the quality of the influent. In terms of flow rates, throughput depends on the hydraulic conductivity of the surrounding media as well as the hydraulic conductivity of the filter fabric. To overcome decreasing flow rates due to clogging over time filters must be maintained or replaced. Inadequately designed filters may lead to clogging, which may cause flooding and subsequent damage to property.

5.3.5.5 Maintenance Considerations

Maintenance requirements for filter fabrics are minimal. In most cases the cost of the filter fabric makes frequent replacement practical. When used in conjunction with media filters, the filter fabric may be inaccessible and may only be maintained during complete system overhaul or during media replacement. For standalone applications, removal of accumulated material on the surface of the filter is adequate maintenance in most cases. Fabrics showing signs of wear or having holes should be replaced to prevent catastrophic failure and subsequent release of previously captured materials. Filter fabric failure may result in the dumping of media or accumulated contaminants into downstream systems possibly resulting in clogging, damage of sensitive downstream BMPs and negative impacts to receiving waters and must be avoided and prevented.

5.3.5.6 Required Surface and Subsurface Area

As previously stated filter fabrics need an external framework for support. Space requirements include the space needed for influent containment and the space needed for the filter fabric supporting framework. A simple application may be a filter fabric wrapped around a perforated outlet pipe in a tank or a vault. In this case the tank provides the influent containment system while the pipe provides filter fabric support. The space requirements for the filter fabric itself is often minimal compared to the space requirements of the other components of the system.

5.3.5.7 Cost Considerations

Filter fabrics are relatively cheap and there is a wide range of products to choose from with new offerings being added with time. Tough, durable fabrics may be economical in some applications while some applications may require biodegradable fabrics.

5.3.5.8 Safety and Aesthetics

Filter fabrics used as components of other BMPs are usually not obvious to the casual observer and therefore do not impact the aesthetics of the system. The primary safety hazard may be clogging in a roadside filter fabric application (e.g., catch basin filter) that causes flooding of the roadway. However, inspection and maintenance according to the manufacturer's recommendations should alleviate flooding concerns for well designed and properly installed roadside applications.

5.3.6 Vegetated Swales and Filter Strips (Biofilters and Bioslopes)

As mentioned previously in Sections 5.1.3, vegetated swales are BMPs and LID practices that typically consist of herbaceous plants and terrestrial grasses planted within a trapezoidal or U-shaped channel to promote shallow, channelized flow (Figure 5-3). As discussed in Section 5.1.5, filter strips are broad, mildly sloped vegetated areas that receive sheet flow from adjacent impervious surfaces (Figure 5-5). Due to the simplistic and flexible design, limited area requirements, ease of maintenance, and demonstrated performance, vegetated swales and filter strips are probably the most common type of stormwater BMP being utilized in both urban, rural and highway environments. They can be used as either primary or secondary treatment system components depending on the target constituents. Also, swales and filter strips offer several options for design enhancements. For instance, swales can be designed as wet systems with a liner (Figure 5-16) and wetland-type vegetation or as dry systems with an underdrain and/or infiltration component. To provide additional or enhanced unit processes some of the other possible modifications include engineered vegetation and underlying soils (e.g., bioretention soils), check dams and berms, and custom inlet and outlet structures (see Section 5.1.3 above for additional . The sections below briefly describe some of the treatment mechanisms and design elements of vegetated swales. The *LID Design Manual* provides additional information and specific design guidance for swales and filter strips (bioslopes).

Figure 5-16. Example of a roadside wet swale planted with wetland vegetation.

5.3.6.1 Associated Unit Processes and Potential Design Enhancements

As the name implies, the primary unit process provided by biofilters is filtration. However, they also provide sorption, sedimentation, and, to a limited degree biological treatment, (i.e., microbial transformation and plant uptake) and volume reduction. Filtration may occur in both the vegetated layer and the soil layer. Thus, it is often recommended to maintain flows at 2 to 3 inches below the height of the vegetation (Horner et al., 1994). Filtration can be improved by having dense vegetation and a granular top soil layer. To improve sorption and filtration, an underdrain may be installed and the top soil layer may be amended with adsorptive media. To improve microbial transformations of dissolved metals or organics, the top soil layer may be amended with a nutrient-rich medium such as compost, but this treatment may be counter productive if nitrogen or phosphorus is a constituent of concern. The hydraulic residence times within biofilters are usually too short for significant plant uptake to occur, so this process, as well as sedimentation, may be improved by installing check dams in swales and minimizing slopes and flow velocities. Depending on the type of vegetation and underlying soils, swales may provide volume reduction via infiltration and evapotranspiration. It should be noted, however, that well-drained soils that are ideal for infiltration will typically only support drought-tolerant vegetation without regular irrigation.

5.3.6.2 Recommended Pretreatment

Vegetated swales and filter strips typically require minimal pretreatment to remain functional. However, high loadings of coarse solids, trash, and debris may flatten grasses and reduce their filtration ability. Thus, trash racks and coarse sediment traps upstream of swales are generally recommended. Biofilters are typically efficient at removing oil and grease, but high loadings may smother and kill vegetation. Therefore, if very high oil and grease loadings are expected, such as from gas stations or high volume roads, an oil-grit separator or absorbent media filter upstream of the biofilter is recommended.

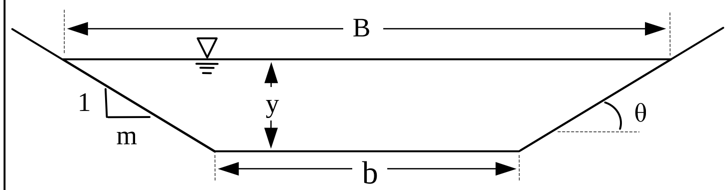
5.3.6.3 Hydraulic Considerations

As a general rule of thumb, biofilters should not have flow velocities greater than 1 to 3 feet per second, depths should be maintained below the height of the vegetation (typically 2-4 inches for swales and 0.5-1 inch for filter strips), and the hydraulic residence time should be no less than 5 to 9 minutes (King County, 1998; Horner et al., 1994). Manning's equation for open-channel flow is the primary design equation used for estimating flow conditions and evaluating design parameters:

$$Q = \frac{k}{n} \cdot A \cdot R^{2/3} \cdot S^{1/2} \qquad [5\text{-}6]$$

Where: Q = the design flow rate (ft^3/s)
k = 1.486 for U.S. customary units (ft and seconds), 1 for metric (meters and seconds)
n = Manning's roughness coefficient, unitless
A = the cross-sectional area (ft^2)
R = the hydraulic radius (A/P, where P = the wetted perimeter) (ft)
S = the longitudinal slope of the channel bottom (ft/ft)

Manning's equation is valid for both channel and sheet flow calculations, and can be used to estimate the design parameters of swales and filter strips. The roughness coefficient, *n*, for swales and filter strips generally range between 0.2 and 0.4 depending primarily on the flow rate, grass height, and blade density (Khan et al., 1992; King County, 1995). The graphic of Figure 5-17 and associated formulas are helpful in making Manning's equation calculations and estimating design parameters.



Useful Geometric Relationships

$A = by + my^2$

$A = By$ (sheet flow)

$B = b + 2my$

$P = b + 2y(m^2 + 1)^{0.5}$

$P = B$ (sheet flow)

$m = \cot(\theta)$

Notation: B = top width, b = bottom width, 1/m = side slope, θ = side slope angle, y = flow depth, A = cross-sectional area, P = wetted perimeter

Figure 5-17. Dimensions of trapezoidal channels and relevant formulae.

5.3.6.4 Maintenance Considerations

The primary maintenance activities for vegetated swales and filter strips are periodic mowing and litter, debris, and sediment removal. Grass clippings should be removed from the site to avoid the release of nutrients during decomposition. Optimal grass height is 4 to 9 inches. For swales with underdrains, periodic inspection during wet weather may be necessary to determine if clogging is occurring. If the biofilter begins to show signs of scour, immediate remedial actions should be taken. This may include installation of straw, coconut fiber or geotextile rolls, or other erosion control fabric and possible reseeding. During vegetation establishment, periodic irrigation and reseeding may also be necessary. The use of fertilizers or pesticides should be avoided.

5.3.6.5 Required Surface and Subsurface Area

Manning's equation should be used to estimate the top width of the channel using the design flow rate and the recommended flow depth. It is generally recommended that side slopes do not exceed 3H:1V (Horner et al., 1994). However, steeper side slopes reinforced with erosion control matting or rip rap may be acceptable. If the bottom width must be greater than about 10 feet to maintain the maximum depth and velocity, a swale divider (berm or partition used to divide the flow in a swale) is recommended. The length is based primarily on the recommended minimum residence time for biofilters of 5 to 9 minutes, but swales should generally not be less than 100 feet in length and filter strips should not be less than 4 feet (King County, 1995).

The subsurface area requirements are only a consideration if an underdrain will be installed. Underdrains are used in areas with low permeability soils underlying the amended soils of the biofilter. Where underdrains will be used sufficient subsurface depth is needed to accommodate amended soils and the underdrain pipe with a sufficient slope to convey the collected infiltration.

5.3.6.6 Cost Considerations

The construction of gassed swales typically requires significant excavation or re-grading while the construction of filter strips may not. Therefore vegetated swales tend to be more expensive than filter strips. Both vegetated swales and filter strips can be easily incorporated into landscaping elements such as open spaces and parking lot stall dividers requiring little additional capital invest. The maintenance and upkeep of biofilters can also be included in general grounds maintenance activities. Capital costs for swales including design and contingency costs have been estimated at approximately \$0.50 per ft^2 (USEPA, 2002b). As compared to other TSCs biofilters are one of the cheapest treatment practices; annual costs are considered to be low.

5.3.6.7 Safety and Aesthetics

Biofilters are probably the most unobtrusive class of treatment system components. Vegetated practices in general tend to look more natural and are more easily camouflaged than non-vegetated practices. Filter strips can be designed and constructed to be indistinguishable from a piece of lawn or road shoulder. Vegetated swales are a little harder to disguise since they typically incorporate a linear channel or depression. Even so, both swales and filter strips are considered to have minimal to no aesthetic impact as compared to other treatment system components. Lack of regular maintenance and a poor choice of vegetation combined with limited pre-treatment and heavy trash and sediment loads can nullify all the aesthetic benefits of biofilters. Fencing or other barriers are rarely needed for roadside biofilter applications if the state DOT standard specifications for shoulder widths and slopes are followed.

5.4 Secondary Treatment BMPs

Secondary treatment BMPs generally target suspended solids. Stormwater constituents with particles sizes ranging from 10 microns to 500 microns are potential candidates for removal by secondary treatment BMPs. A discussion of the associated unit processes and the factors that enhance or hinder these processes for each of the secondary treatment BMPs is provided below.

5.4.1 Bioretention

Bioretention BMPs utilize vegetation and permeable soils to promote biological processes, infiltration, and stormwater retention/evapotranspiration to remove pollutants and reduce stormwater runoff volumes (USEPA, 1999g). Bioretention areas are a key element of LID due to their capability to control runoff volumes through infiltration and evapotranspiration. Bioretention areas are designed to incorporate many of the treatment mechanism present in forested ecosystems including filtration, infiltration, and biological treatment process (WADOT, 2003). Vegetation also has indirect treatment functions including:

- Reduction of flow rates allowing for filtering and settling of particulates, and more time for microbial contact with pollutants
- Transpiration
- Erosion control
- Support of the microbial community by supplying a substrate, aerating the root zone, forming symbiotic associations, and providing organic litter.
- Improved infiltration into the shallow soils promoted by stem and root intrusion

Figure 5-18 illustrates two bioretention areas placed in parking lots. Similar designs may be installed near roads and beneath downpouts of buildings.

Figure 5-18. Example of parking lot bioretention areas.

It is important to select plants with characteristics that support these functions. Always consult a qualified botanist, landscape designer, and/or landscape architect regarding plant selection before installing a vegetated system. Useful characteristics of vegetation for stormwater treatment systems include:

- Tolerant of site-specific and climatic conditions (e.g., drought-tolerant plants).
- Not invasive or noxious.
- Tolerant of typical stormwater pollutant concentrations.
- Can uptake, store, or otherwise remove pollutants.
- Easy to establish and resilient to stress.
- Low maintenance requirements (e.g., minimal irrigation and fertilization).
- High growth rates, large surface area of roots, stems and leaves.
- Salt-tolerant in areas with high concentrations of soluble salts (arid regions), or cold climates where deicing agents are used.
- Supports symbiotic associations with microbes.
- Plants are easily obtained, affordable, and aesthetically pleasing.

5.4.1.1 Associated Unit Processes and Potential Design Enhancements

Vegetation can remove dissolved metals and organics through direct uptake and storage, and excretions from plant roots assist in metal precipitation. The vegetation reduces flow velocities allowing particulates to be removed by sedimentation. Vegetation increases microbial populations, which transform and/or remove a variety of organic and inorganic constituents. Plant detritus provides sorption sites for various organics and metals.

Metal stabilizing plants limit the mobility and bioavailability of metals in soil by sorption, precipitation, complexation, or reduction of metal valences. The ability of these plants to sequester metals in the root zone is a defense strategy to prevent uptake of toxics into above ground biomass. However, plants need to be able to tolerate high concentrations of metals in the roots. Metal stabilizing plants have extensive roots systems that grow rapidly. This characteristic is found in many herbaceous species, grasses, trees, and some wetland plants.

In cold climates, plants have a shorter growing season and may go dormant in the winter. Uptake of nutrients into plant materials only occurs during the active growing season, and nutrient concentrations in vegetation are highest early in the growing season; seasonal changes in plant tissue concentrations tend to vary by nutrient and by plant species (Vymazal, 1995).

Native plants are preferred as they are typically better suited to the local climate, easier to establish than exotics, and generally provide the highest benefit to the local ecosystem. Many native plants have local specificity, which is relevant if a vegetated treatment system is located near open space; in this case, local plant varieties should be used whenever possible to prevent genetic mixing and decline of local plant communities. Site characteristics such as soil type, inundation patterns, topography, sun exposure, and ecological succession affect the plants that can be used at a site, the planting methods that should be used to improve plant establishment, and long-term establishment of plants. Where available, use local guidance on plant selection such as municipal stormwater design manuals or local conservation groups. Other design considerations for bioretention areas include:

- Preserve existing natural vegetation whenever possible.
- Diversify plant species to improve wildlife habitat and minimize ecological succession.
- Situate plants to allow access for structure maintenance.
- Avoid plants with deep taproots if appropriate, as they may compromise the integrity of filter fabric or subsurface drainage facilities.
- Use seed mixes with fast germination rates under local conditions planting at appropriate times of the year.
- Temporarily divert flows from seeded areas until vegetation is established.
- Stabilize water outflows with plants that can withstand strong current flows.

Various municipal stormwater design manuals (e.g., Prince Georges County, 2001; Puget Sound Action Team and WSU, 2005) provide recommendations on plant species suitable for the region, including natives. Local conservation groups may also provide information. Nonlocal information should be used with caution. For example, lists of plants commonly used in stormwater treatment systems often contain species that would be considered invasive in other areas. Knowledge of species that exist locally in natural upland and wetland areas is also useful for selecting native plants. Many native plants have local specificity, which is relevant if a vegetated treatment system is located near open space; in this case, local plant varieties should be used whenever possible to prevent genetic mixing and decline of local plant communities.

Some general internet resources for native plants include:

- Lady Bird Johnson Wildflower Center Native Plant Information Network, http://www.wildflower2.org
- U.S. EPA Green Landscaping Resources, http://www.epa.gov/greenacres/resources.html#Natural%20Landscaping
- FHWA Roadside Use of Native Plants, http://www.fhwa.dot.gov/environment/rdsduse/index.htm
- USDA PLANTS Database, http://plants.usda.gov

Some general internet resources for invasive plants include:

- National Park Service Plant Conservation Alliance. Fact Sheets: Alien Plant Invaders of Natural Areas, http://www.nps.gov/plants/alien/factmain.htm#pllists

- National Invasive Aquatic Plant Outreach and Research Initiative, Non-Native Invasive Aquatic and Wetland Plants in the United States, http://aquat1.ifas.ufl.edu/seagrant/aquinv.html

Resources for removal of organic and inorganic compounds by plants include Environment Canada's two phytoremediation databases: PHYTOPET (for petroleum hydrocarbons and related organics such as PAHs and VOCs) and PHYTOMET (for 19 metals). PHYTOPET can be accessed on the Web at http://www.phytopet.usask.ca/mainpg.php, and PHYTOMET is available on CD-ROM. The databases are searchable by plant, pollutant, phytoremediation mechanism, and phytoremediation potential. The databases include over 775 terrestrial and aquatic plants.

5.4.1.2 Recommended Pretreatment

Bioretention areas require minimal pretreatment to operate properly when treating urban or highway stormwater runoff. Bioretention may not be suitable for areas where spills frequently occur. The removal of trash and debris is beneficial to reduce maintenance costs and preserve aesthetics by collecting this material near the entry point to the bioretention area with trash racks. Bioretention treatment is usually effective at removing oil and grease, but high loadings can smother and kill vegetation. If very high oil and grease loading are expected, such as from gas stations or high volume roads, an oil-grit separator or absorbent media filter upstream of the biofilter is recommended.

5.4.1.3 Hydraulic Considerations

Bioretention areas should be designed as a volumetric BMP with the consideration that a flow component (infiltration) may influence the rate at which water is stored in the bioretention area. Bioretention areas are typically sized to store runoff from the design storm within the soil matrix of the BMP. Sizing a bioretention area should account for irrigation of the vegetation once soil moisture reaches a certain level, which reduces the effective porosity of the soils somewhat for capture of stormwater runoff.

Soils with high infiltration rates are most suitable for bioretention areas. Some ponding over the bioretention area is desirable to store runoff in those instances when the runoff rate exceeds the rate of infiltration. A commonly used maximum ponding depth is 6 inches, deep enough to provide storage capacity for high rainfall intensities but shallow enough that standing water will only occur for a few hours typically.

Since bioretention areas are offline facilities, the main hydraulic concern for a bioretention area is the erosion potential of the incoming runoff. An energy dissipation device should be placed at the inlet to prevent any erosion and to dissipate flows so that flow through the bioretention BMP is relatively uniform and does not channel. A secondary consideration is the depth to groundwater and infiltration of the native soils. Areas with high groundwater (e.g., 6 feet or less) are not suitable for bioretention to prevent groundwater contamination. If bioretention areas are used in areas with poorly drained soils (e.g., infiltration less than 0.5 inches per hour) the soil within the BMP should be amended to provide sufficient infiltration rates, and the design should include an underdrain to collect infiltrated stormwater.

5.4.1.4 Maintenance Considerations

Activities associated with the establishment and maintenance of vegetation may include:

- Irrigating, particularly after initial installation of plants. Some plants may require dry season irrigation.
- Monitoring changes in site conditions that affect vegetation and status of vegetation (a minimum of annual inspections should be conducted).
- Controlling weeds, diseases and insects (use of most pesticides is not encouraged). Integrated pest management (IPM) approaches are highly recommended.
- Mulching at least annually to suppress weeds, increase moisture retention, and replenish organic matter.
- Removing and replacing dead and diseased vegetation.
- Removing species as needed to prevent undesirable ecological succession.
- Pruning woody vegetation as needed to avoid conflicts with overhead utilities or hazards with adjacent people and property.
- Removing plant stakes after the first growing season.
- Removing litter and debris for water pollution control, and to reduce favorable environments for harmful insects and pathogens.
- Applying fertilizers and soil amendments as needed.

5.4.1.5 Required Surface and Sub-surface Area

Bioretention BMPs are usually sized in a manner similar to detention ponds to meet the requirements of the regulatory agency. The footprint area requirements are usually somewhat larger than requirements for detention ponds. The tributary area to the bioretention BMP should be about 1 acre or less depending on the expected rainfall characteristics. Dimensions should be about twice as wide (perpendicular to the direction of inflow) as they are long to promote flow spreading and minimize the chances of concentrated flow. The depth of amended soil within the BMP should exceed the maximum anticipated root depth of the vegetation; four feet is commonly used.

5.4.1.6 Cost Considerations

Costs are associated with planting, monitoring and maintenance (USACE, 2000). Planting costs may include labor, seed, contract-grown materials, nursery materials, equipment rentals (e.g., bulldozer, rototiller), and soil amendments. Costs of actual plant materials are highly variable and are affected by several factors. Seeds are much less expensive than transplants. Containerized/balled and burlapped stock is more expensive than bareroot stock. Topsoil seedbank materials are less costly than transplants. Species that are difficult to propagate, in short supply, in short demand, or require specialized handling techniques are often very expensive. Mechanized planting typically costs less than manual planting, particularly on large sites, and planting under water is much more expensive than planting on upland sites.

Maintenance costs may include soil amendments, irrigation, replacement or supplemental plants, and control of undesirable vegetation, insects, and diseases. Use of natives may reduce replacement costs because natives have a higher survival rate than exotic plants.

5.4.1.7 Safety and Aesthetics

Bioretention areas can be integrated virtually seamlessly into roadside landscaping and therefore present few vehicle safety hazards. The use of large diameter trees (>4 in.) or other fixed-object hazard should be avoided if the bioretention area is located near the roadway. State DOT highway design manuals have guidance on the minimum buffer distance to fixed objects or the need for guardrails. Maintenance personnel performing routine maintenance, such as pruning or mowing, will need adequate space for safe access and operation of vehicles and equipment. Oncoming traffic should be forewarned through the use of flaggers, cones, or signage whenever roadside maintenance is being performed.

Through the use of a diverse selection of plants, trees, and shrubs, the aesthetics of a roadside bioretention area can be greatly enhanced. A good landscape design can enhance adjacent property values. Aesthetic considerations for vegetated systems include:

- Morphology (e.g., size, shape, and growth habit) and color of foliage, flowers, fruit, stems and bark.
- Seasonal characteristics (e.g., deciduous or evergreen).
- Ability to attract birds, butterflies, and beneficial insects. However for applications close to high speed traffic, the use of colorful flowers or plants that attract wildlife may be undesirable.
- Desired landscaping levels are obtained upon completion of construction or shortly thereafter.

5.4.2 Media Filters

Stormwater is captured and directed through a medium such as sand, compost, zeolite, etc. either under gravity or pressure flow in a media filtration system. Media filtration is primarily intended to treat fine particulates and associated pollutants. Depending on the type of media they may also be used for enhanced treatment of dissolved constituents including dissolved metals and nutrients. Sand filtration is a widely used method of stormwater treatment, with designs such as the Austin, Texas Filter having a history of successful usage throughout the United States (Figure 5-19). Other media such as zeolite, peat moss, compost leaves, and various sorbent materials are also utilized for supplemental stormwater treatment within devices such as catch basin inserts, oil/water separators and some proprietary devices (e.g. StormFilter™ and AquaFilter™). Figure 5-20 is an example surface installation of a StormFilter™ Vault.

The choice of filtration medium is important in the design of a filtration system. Sand is often used in filtration devices due to many factors including cost, availability, handling and maintenance issues. Sand filters are normally inert media filters (not engineered/pretreated chemically). While they remove solids well, they are least effective in removing organics and dissolved constituents. In both wastewater and stormwater treatment engineered media such as iron and aluminum hydroxy coated sand media have been shown to be highly effective in removing dissolved metals (Sansalone, 1999; Benjamin et al., 1996) and dissolved phosphorus (Ayoub et al., 2001). In water and wastewater treatment, slow sand filtration has shown to be extremely effective in removing suspended particles, bacteria, viruses, and *Giardia* cysts. Microbes on the surface of filtration media provide an important treatment pathway for many pollutants. Bacteria and algae utilize nutrients for their life functions thereby removing them from the stormwater stream. Biological trickling filters are very effective at removal of BOD, nutrients and other pollutants found in wastewater. However, because biological filters require steady loading rates and generally do not cope well with extreme flows, there is a problem adapting this technology for stormwater treatment, because stormwater flow is rarely consistent. In times of extreme dry period the microbes present on the surface of the media would die off. This die-off would reduce the system's ability to treat the next storm event, and depending on the life cycle of the bacteria, may result in the microbes releasing stored nutrients, exacerbating nutrient levels in the stormwater stream.

Sand filters are also limited in their ability to remove organics and dissolved metals. Composted leaf filters that are made of yard waste, primarily leaves, have been found to be effective in removing oils, greases, organic pollutants, and nutrients due to their high humic content and sorption capacity. Media such as activated carbon, peat, zeolites, activated alumina, polymer and synthetic resins are widely used for treatment of dissolved metals and/or organics in water and wastewater treatment. The potential use of wood and other types of fibers known for their significant sorption capacity for metals has been investigated as filtration media for stormwater by the USDA-FS Forest Products Laboratory, Madison, Wisconsin (Han et al., 1999).

Figure 5-19. Example Austin Sand Filter.
http://www.stormh2o.com/images/sw_0103_p67_no.%2012.jpg

Figure 5-20. Example StormFilter™ Vault

5.4.2.1 Associated Unit Processes and Potential Design Enhancements

Treatment of stormwater by media filtration primarily involves the following processes:

- entrapment/straining of solids by the media;
- sorption, adsorption and ion exchange of pollutants depending on the media type; and
- microbial degradation on the surface of the media.

These and other relevant physical, chemical, and biological mechanisms that assist treatment by filtration are discussed in detail in Sections 4.2 to 4.4. Treatment by media filters occurs through sorption or adsorption of pollutants to the medium itself or biological utilization of the pollutant by microbes on the medium surface. Sorption to media particles is limited by the physical and chemical properties of the medium, surface area per unit volume and most importantly by the fraction of bonding sites utilized, i.e., the amount of material already sorbed to the filter medium. When pollutant are removed by reactive mechanisms such as cation exchange, precipitation, chelation, or adsorption it should be understood that these reactions have limits established by sorption capacity of the media and reaction kinetics. For example, the sorption capacity of the media is usually given as mg of adsorbent per kg of media. Given the mass of the media, the potential mass of pollutant removal can be calculated and compared with the influent pollutant loads. Reaction kinetics can cause a decrease of the pollutant removal rates as saturation increase and/or the pollutant concentration decreases. The media should be periodically replenished to maintain the performance effectiveness of the media.

5.4.2.2 Recommended Pretreatment

All media filters require some form of pre-treatment to avoid clogging. In many sand filters an upstream sedimentation tank is designed immediately upstream of the filter to remove coarse sediment and distribute the flow evenly across filter. Hydrodynamic devices such as a CDS (proprietary Continuous Deflection Separator) unit would also be suitable, provided they are sized to remove coarse sediment. Without pretreatment sand filters tend to clog, and their effectiveness is decreased. Filters should also be designed with provision for overflow or bypass for extreme storm events to prevent damage and to prevent flooding if the device is blocked.

5.4.2.3 Hydraulic Considerations

Media filters are flow based treatment systems. Relevant hydraulic principles that govern water flow through a porous medium are provided in Section 5.3.5.4 with regard to filter fabrics. Sizing and design of a media filter typically involves determining the water quality volume of the pretreatment unit and determining the surface area of the filter. The filter area for sand and organic filters should be sized based on the principles of Darcy's Law (Equation 5-4). The required filter bed area can be computed based on the City of Austin (1996) design manual:

$$A_f = \frac{WQ_v \cdot d_f}{k \cdot (h_f + d_f) \cdot t_f} \qquad [5\text{-}7]$$

Where: A_f = surface area of filter bed (ft²)
WQ_v = water quality design volume (ft³)
d_f = filter bed depth (ft)
k = hydraulic conductivity of filter media (ft/day)
h_f = average height of water above filter bed (ft)
t_f = design filter bed drain time (days)
(1.67 days or 40 hours is recommended maximum for sand filters, 48 hours for bioretention)

To ensure that the system could convey the peak design flow a backwater calculation accounting for media losses, pipe losses from the point of downstream control should be performed. The velocity of the pipe discharge onto the filter bed is important as it should not scour or dislodge the filter media.

The total flow rate Q through the media is given by:

$$Q = qA \qquad [5\text{-}8]$$

Where: q = specific flow rate for the medium (gpm/ft²)
A = area of the filter bed (ft²)

For proper filter design the specific flow rate of the media should match the design flow rate for the system to function effectively. The thickness of the media along with the specific flow rate determines the length of time (residence time) the water has to be treated. The longer the residence time the more effective the pollutant removal will be, especially for dissolved constituents treated with reactive media.

Sand filters require a significant amount of hydraulic head, usually about 4 ft, to allow flow through the system. However, a perimeter sand filter would require only about 2 ft of head. Hydraulic conductivities (k) values used in Equation 5-7 depend on the media used. Typical k values are 3.5 ft/day for sand, 2.75 ft/day for peat/sand mixture and 8.7 ft/day for compost leaves (CASQA, 2003).

Media filtration units are designed with overflow structures and flow spreaders to prevent damage under high flow conditions. Filtration units are often slow working and require a large head loss to drive water through the media. Other factors such as keeping microbial organisms on the surface of the media viable during dry conditions are also a problem with media filtration. For an Austin Sand Filter the underdrain piping should consist of a main collector pipe and two or more lateral branch pipes, each with a minimum diameter of 4 inches. The pipes should have a minimum slope of 1% (1/8 in/ft) and the laterals should be spaced at intervals of no more than 10 feet.

5.4.2.4 Maintenance Considerations

Long term performance of media filtration is dependent on regular system maintenance with the media maintained in a working condition, avoiding piping bypass, and replacing the entire media when necessary. Replacement of the media is a problematic and potentially hazardous operation. Depending on the filter operation duration and the pollutant loading rate of the contributing catchment, the retired media may be classified as a hazardous waste and therefore must be properly handled and disposed.

Sand filters need to be monitored on a regular basis and after every storm event for:

- Ponding, clogging and blockage of the filter media.
- Depth of sediment in the settling tank/pretreatment device.
- Blockage of the outlet from the settling tank/pretreatment device to the filter.

The following activities may be required for maintenance of stormwater media filters:

- The sediment and litter should be removed from the pretreatment device.
- The filter surface could be regularly raked to remove sediment and to break up any crusts (to improve infiltration).
- The top layer of the filter media can be removed and replaced.

If the filter is not cleaned frequently, the entire filter media may need to be replaced due to migration of sands within the media. This can result in frequent maintenance being more cost effective in the long term.

Contaminated sand and other material removed from the filter or the sedimentation chamber (if applicable) can be removed to a landfill or appropriate disposal facility.

5.4.2.5 Cost Considerations

Filtration devices are usually expensive as they are usually made in-situ. Maintenance costs usually are also high due to the amount of work and due to the confined spaces that maintenance staff may be required to work in. The EPA fact sheet indicates a cost for an Austin Sand Filter at $18,500 for a 0.4 hectare (1-acre) drainage area. However, the same design implementation at 1.1 ha site by the California Department of Transportation, cost $240,000 (CASQA, 2003). Consequently, there is no set cost per hectare available for media filtration units due to the variability of construction costs, cost of the media and size of design storm event chosen for treatment.

5.4.2.6 Safety and Aesthetics

Safety and aesthetics is generally not an issue for underground structures. However if the filter media become anoxic, foul odors may be emitted due to the reduction of sulfate to hydrogen sulfide. Also, if the filter has a settling basin or available surcharge depth of greater than 4 feet, fencing or other barrier is recommended. If the access to the site is near the roadway, appropriate safety measures (e.g., lane closures, cones, warning signs, flaggers, etc.) must be taken according to the specific safety requirements of the state DOT. If standing water is likely to be present greater than 72 hours, a mosquito control program should be developed.

5.4.3 Infiltration/Exfiltration Trenches and Basins

Infiltration and exfiltration BMPs provide volume reduction by providing a means for influent stormwater to infiltrate into the ground. In addition to volume reduction, infiltration and exfiltration trenches utilize the biological and chemical processes that occur in soils to mitigate stormwater constituents. Section 5.1.2 briefly discussed roadside infiltration and exfiltration trenches and provided an example photograph. Figure 5-21 is another example of an infiltration trench used to treat residential runoff. Section 5.6.8 also discusses infiltration trenches in the context of hydrologic control.

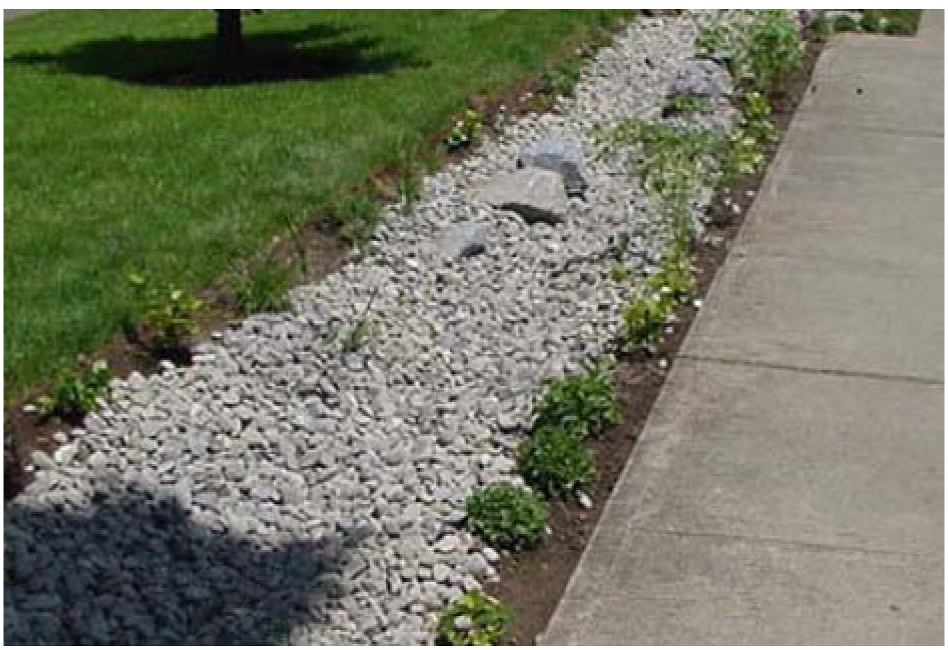


Figure 5-21. An Example of an Infiltration Trench.
Source: BES 2002.

5.4.3.1 Associated Unit Processes and Potential Design Enhancements

The treatment processes that most commonly result in water quality improvements of stormwater in infiltration/exfiltration BMPs include the following:

- Filtration or entrapment/straining of solids as they pass through media particles;

- Sorption, adsorption and ion exchange of pollutants which is a function of media type;
- Reduction in pollutant loads from infiltration of stormwater.

The pollutant reduction processes that occur in infiltration/exfiltration BMPs are very similar to the processes that occur in media filters since infiltration/exfiltration BMPs can be considered as media filters with the added benefit of infiltration processes. As with media filters the sorption capacity of infiltration/exfiltration systems depend on the physical and chemical properties of the media, pollutant loading rates and the length of time the media has been in use in the facility. Periodic replacement of the media is recommended to maintain pollutant reduction and infiltration efficiency. Like most infiltration practices, the addition of vegetation prevents the formation of crusts on the surface of infiltration/exfiltration BMPs. The addition of vegetation also provides plant uptake and storage as an additional pollutant reduction mechanism.

5.4.3.2 Recommended Pretreatment

Infiltration/exfiltration BMPs are prone to clogging if the proper pretreatment is not provided. Upstream sedimentation facilities help remove coarse sediment that otherwise can expedite clogging. Other sediment and floatables removal devices such as CDS units and public works practices can also provide pretreatment. In addition to these pretreatment devices, relatively cheap screens and racks can be added where applicable to keep out large objects that may damage media beds and inlet and outlet structures.

5.4.3.3 Hydraulic Considerations

Infiltration/exfiltration BMPs are primarily flow based BMPs; however, infiltration basins may have enough built in storage to capture storms with higher peak flows than the system infiltration rate thereby acting like volume based BMPs. Storage typically consists of pore spaces within the media and the space above the media which depends on the allowable ponding depth. Exfiltration trenches on the other hand are typically flow based only due to less built-in surface storage. The reader is referred to the Hydraulic Considerations discussion under Section 5.4.2 - Media Filters for more information.

5.4.3.4 Maintenance Considerations

Periodic maintenance is vital for the efficient long term performance of infiltration/exfiltration facilities. Typical maintenance includes removal of deposited materials, tilling of the media bed to break surface crusts, maintenance of media bed vegetation if present, and regular inspections for clogging and damage to berms and inlet/outlet structures. Lack of regular maintenance may eventually compel the replacement of the filter media due to migration of fine particles in the filter bed and/or the loss of pollutant removal properties of the media. Contaminated media and captured material may be safely disposed of at a landfill.

5.4.3.5 Required Surface and Subsurface Area

The surface space requirements of infiltration facilities depend on the design flow rate, the allowable depth of ponding, and the permeability of the entire system. Impermeable soils may result in facilities that have large surface areas while facilities located at more permeable sites require smaller surface areas to operate at the same flow rate. In the same way, facilities that have built-in storage allow ponding that can attenuate peaks or store incoming flows to be infiltrated over time at convenient flow rates, while facilities that have less built-in storage can only accommodate flows up to the infiltration rate of the facility. Subsurface space requirements depend on the porosity of the media and the design storage capacity, while the required surface area and depth of the facility are a function of the required volume. Large subsurface volumes also provide storage of incoming flows and contribute to the total built-in storage of the facility with the same benefits as previously discussed. One estimate of the space requirements for infiltration basins is about 2% to 3% of the tributary area (CASQA, 2003).

5.4.3.6 Cost Considerations

Infiltration/exfiltration BMP costs are moderate compared to other BMPs depending on land costs. Infiltration/exfiltration trenches may be cheaper and easier to construct than infiltration basins which can require significant excavation. Maintenance costs of infiltration facilities in general tend to be higher than maintenance cost for other BMPs such as dry detention ponds. One estimate of the construction costs of infiltration basins is about $2 per ft of storage for a 0.25-acre basin (1991 dollars) while maintenance costs are estimated to be about 5% to 10% of construction costs (CASQA, 2003).

5.4.3.7 Safety and Aesthetics

Infiltration/exfiltration basins and trenches can be vegetated and easily disguised to blend with landscaping elements and are therefore considered to have a low impact on safety and aesthetics. However inadequate pretreatment and less than frequent maintenance may result in the degradation in aesthetics from accumulated sediment, trash and floatables, and may present a safety hazard if flooding occurs. If the access to the site is near the roadway, appropriate safety measures (e.g., lane closures, cones, warning signs, flaggers, etc.) must be taken according to the specific safety requirements of the state DOT.

5.4.4 Detention and Retention Ponds

Detention and retention BMPs are widely used practices that provide both peak attenuation and water quality benefits. Depending on the components included in these systems, a host of unit processes can be attained including peak attenuation, flow control, sedimentation, irradiation (UV exposure), microbially-mediated transformations and plant uptake.

Detention and retention ponds are one of the most commonly used stormwater management practices (USEPA, 2002a). Detention ponds are designed to completely drain between storm events, while retention ponds have a permanent water quality pool. Detention ponds may thus also be known as “extended dry detention” or “dry ponds” because of their characteristic dry-down between storm events. The term detention and retention pond as used in this discussion will be limited to the open natural basin that acts as the detention component in detention ponds or the retention component in retention ponds. Please note that this does not include sediment forebays or inlet and outlet structures that are normally considered integral parts of a detention or a retention pond. The primary treatment mechanisms observed in detention and retention ponds are typical provided by two main unit processes: sedimentation and volume reduction.

The popularity of ponds stems from their demonstrated ability to function as water quality improvement BMPs while providing flood control through controlled release of collected water. Detention ponds can also provide multiple uses such as play fields, parks and recreational areas; retention ponds can often be transformed into stormwater treatment wetlands with little additional cost (VSC, 1999) and often have the aesthetic values of miniature lakes. Figure 5-22 is an example of a retention pond used for stormwater treatment (photo taken from spillway).

Figure 5-22. An Example of a Retention Pond.

5.4.4.1 Associated Unit Processes and Potential Design Enhancements

The main pollutant removal mechanism in ponds is provided by the sedimentation unit process. The extent of sedimentation that occurs is dependent on the residence time. Typical residence times for detention ponds are 24, 48, and 72 hours with variable or constant draw-down rates. The addition of extended detention/retention pools can increase overall treatment by providing increased detention time for low flows. A similar effect is achieved by using variable drawdown times whereby the storage volume in the bottom half of a pond is detained longer than the top half to provide longer detention of runoff from small storms that do not completely fill the pond.

The efficiency of the sedimentation process depends on the properties of stormwater flow through the pond. Short circuiting and re-suspension of previously captured sediments can lower the overall efficiency of a pond and should be prevented through sound design, such as baffles and relative location of the inflow and outflow structures (should be far apart). Under ideal conditions the flow through a pond would be plug flow, where slugs of water move through the pond in a sequential manner with little or no mixing. Short circuiting results in dead zones in the pond where pockets of water get trapped in quiescent areas, taking up space and forcing subsequent slugs of water to take shorter paths through the pond. Selection of pond length-to-width ratios of greater that 2 (CASQA, 2003; USEPA, 1999e) minimizes the incidence of short circuiting. Frequent maintenance, custom inlet and outlet structure design and positioning of inlets and outlets above the pond bottom to provide space for captured sediment all serve to minimize the frequency of occurrence of re-suspension (see Section 4.2.3). Other pollutant removal operations include natural flocculation (Section 4.4.2) and various biological processes (Section 4.3). While both detention and retention ponds are often vegetated, the biological activity in retention ponds usually exceeds that of detention ponds due to the permanent pool and emergent wetland vegetation typically planted around the pond perimeter and littoral zone.

In addition to pollutant removal, ponds also serve as excellent hydrologic control components by providing peak attenuation and flow control through storage, infiltration, and evaporation. Infiltration is more likely to occur in dry ponds, particularly if the pond is constructed with a permeable bed. It must be noted that in some areas there may be treatment requirements prior to infiltration that may apply for ponds with a significant infiltration component. The evaporation losses realized for a pond mainly depend on the surface area of the pond, wind speed, and the relative humidity. In general ponds with larger surface areas will evaporate more. The surface area of a pond can be increased by making the pond shallower while providing the same volume. Evaporation can also be enhanced by adding sprinklers or fountains or other such devices that increase air to water contact.

Although both evaporation and infiltration provide volume reduction, infiltration has the additional benefit of also providing pollutant load reductions. Evaporation typically leaves stormwater constituents in the pond as the water moves to the gaseous phase while infiltration removes both the water and its dissolved constituents from the pond (see Section 4.1).

5.4.4.2 Recommended Pretreatment

Presettling basins or sediment forebays are the most commonly used pretreatment practices for ponds (Sections 5.2.2 and 5.3.4). This form of pretreatment is so prevalent that the sediment forebays and the retention or detention pond are often considered as a unit. The primary purpose of the sediment forebay is settling large solids and coarse sediment entering the system. The sediment bay also serves to attenuate the turbulence and the energy of the influent flow stream while providing a uniform more streamlined flow profile into the pond. Other storage BMPs can be used as replacements for the sediment forebay; for example, biofilters can also be used as pretreatment for ponds for relatively low flow rates. Trash racks and screens are not recommended as standalone pretreatment devices for ponds; however, the selected pretreatment should incorporate trash racks or screens to keep large objects out of the system. Large objects could potentially damage inlet and outlet structures and cause re-suspension of captured sediment.

5.4.4.3 Hydraulic Considerations

The pond bottom must be at least 0.5 feet (King County, 1998) below the outlet to provide room for sediment storage. Embankments must be structurally sound to withstand hydrodynamic pressures at depths equivalent to the pond bottom depth. The ground water level must be below pond bottom for dry ponds, otherwise the effects of buoyancy must be considered in the overall pond design. Overflow structures and spillways must be designed according to applicable design requirements (typically required to safely pass the 100-year flow) to protect against embankment breaching (King County, 1998).

A summary of some of the important geometric design parameters is presented in Figure 5-23. Most jurisdictions have freeboard requirements of about 1-foot to 2-feet. Freeboard provides an extra margin of safety during large events and is recommended for every pond design. Please note that other treatment system components that are closely associated with ponds such as outlet/inlet structures, sediment forebays and spillways have been excluded from the discussion.

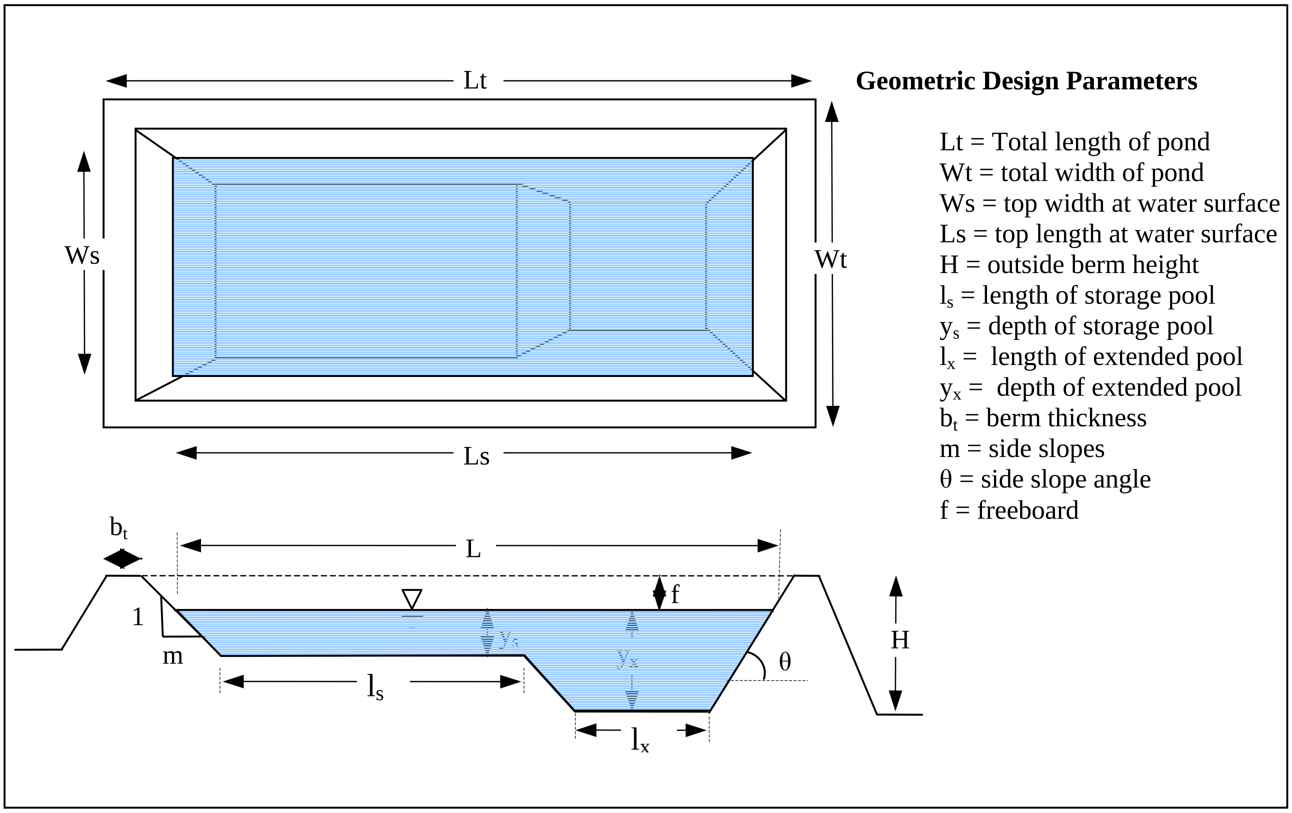


Figure 5-23: Important Geometric Design Attributes for Ponds

5.4.4.4 Maintenance Considerations

The primary maintenance activities for ponds include removal of inlet and outlet obstructions after large runoff events, maintenance of vegetation, periodic removal of accumulated sediment, periodic repair of pond berms, maintenance of access roads, and repair of spillways and inlet/outlet structures. Regular inspections and inspections after large rainfall events are recommended to detect maintenance and repair needs as early as possible. The importance of maintenance cannot be over emphasized for proper BMP function. Pond maintenance affects hydraulic performance as well as aesthetics (see Hydraulic Considerations and Safety and Aesthetics sections).

5.4.4.5 Required Surface and Subsurface Area

Detention and retention ponds must be sized to meet the requirements of the governing regulatory agency. Most jurisdictions provide methods for sizing the design volume. These are sometimes as simple as "capture the first half-inch of runoff from the directly connected impervious area," but much more sophisticated methods may be applied (Chapter 7). Using the design volume, a maximum allowable depth, and an acceptable length to width ratio the required surface area can be easily calculated using geometric relations that are applicable for the selected cross sectional shape of the pond. Some of the frequently used geometric relations pertinent to trapezoidal cross-sections are shown in Figure 5-23. The surface area requirements for detention and retention ponds range from medium to high and can be manipulated by varying the depth of the pond within the accepted range of depths (a maximum depth of only 4 feet is typical).

5.4.4.6 Cost Considerations

The capital costs for detention and retention ponds range from medium to high with costs typically being higher for retention than detention ponds. The addition of multi-stage outlet structures for variable draw-down times, spillways, maintenance access ramps, multiple interior berms and extended detention/retention pools can significantly increase construction costs for retention and detention ponds,. The addition of impermeable liners in the case of retention ponds can also impact capital costs. 6.9.)

5.4.4.7 Safety and Aesthetics

Well designed and maintained retention ponds can enhance the aesthetics of the surrounding area. The aesthetic appeal of open water areas is a well know and often utilized feature in the field of landscape architecture. However, poorly designed and maintained ponds can develop unpleasant odors or breed disease vectors and lower neighboring property values. Pond designs that consistently apply the techniques presented in this section are more likely to improve neighboring property values.

Opportunities abound for improving pond aesthetics. Pond berms or embankments can be planted with grasses and shrubs and routinely manicured. Wet ponds can support various kinds of aquatic plants and emergent wetland vegetation. Fountains and sprinklers, which increase evaporation and increase aeration, can be installed in retention ponds. An aesthetically pleasing design combined with public safety elements such as unobtrusive fences (e.g., wood as opposed to chain-link) and signage serves to increase public acceptance of pond-type BMPs. Fences serve to limit access and protect the public (little children) while keeping vandals out of the pond site. Regular inspections and routine removal of accumulated sediment can mitigate odors and vectors. The breeding of mosquitoes, which is an aesthetic detractor as well as a human health hazard, can be controlled using a variety of methods, such as *Gambusia affinis* (mosquito fish) and *Bacillus thuringiensis israelensis* (Bti). Appropriate safety measures (e.g., lane closures, cones, warning signs, flaggers, etc.) must be taken according to the specific safety requirements of the state DOT, whenever the site must be accessed for inspection, monitoring, or maintenance.

5.5 Tertiary Treatment BMPs

Tertiary BMPs typically provide dissolved constituent and pathogen removal. For each of the BMPs discussed in this section, a brief description of the treatment mechanism is discussed in addition to the unit processes that occur in the BMP. A discussion of the factors that hinder or favor the incidence of the unit processes in each BMP is also presented. It is important to note that limited data on the tertiary treatment of stormwater, particularly some of the more advanced innovative treatment processes typically only applied to municipal water treatment operations.

5.5.1 Advanced Biological Systems (Wetlands, Bioreactors, Etc.)

Advanced biological systems can be defined to include a wide range of practices from treatment wetlands to vegetated swales to biological reactors commonly used in waste water treatment plants. Biological reactors such as trickling filters or biological contactors are rarely used for stormwater treatment and are outside the scope of this document; for further detail the reader is referred to references such as Metcalf and Eddy, 2003.

Treatment wetlands and bioretention systems are well suited to stormwater treatment. Bioretention systems remove pollutants primarily through physical processes with biological reactions providing secondary benefits. Bioretention systems are discussed in Section 5.4.1 of this document; this section describes treatment wetlands. Treatment wetlands are designed for water quality treatment with the aesthetics a secondary benefit. They make use of processes that occur in natural wetlands as well as in conventional wastewater treatment plants, but are "simpler" than conventional technologies because they do not require advanced containment and control systems. A main drawback of treatment wetlands is that they require comparatively large areas of land and at least a minimal year-round flow of fresh water. Figure 5-24 shows examples of constructed wetlands used for stormwater treatment.

Treatment wetlands can be classified as surface or subsurface wetlands. Surface wetlands typically have a soil bottom, with emergent vegetation, and a water surface exposed to the atmosphere. Water moves through the wetland at low velocities in a quiescent manner. The plants are adapted to continuously saturated soil conditions and anaerobic soils. Surface wetlands have variable oxygen levels that are highest at the air/water interface and decrease with depth. Surface wetlands are generally the least costly to construct (on a per acre basis) and simplest to design. Surface wetlands provide greater flow control, and have secondary benefits of aesthetic appeal, wildlife habitat availability, and recreational opportunities (ITRCWT, 2003).

Subsurface flow wetlands are usually constructed with porous material such as soil, sand, or gravel for a substrate and are designed so that water flows through the substrate below ground surface without exposure to the atmosphere. Advantages of subsurface flow wetlands include increased treatment efficiency, reduced land area requirements, fewer pest problems, reduced exposure to pollutants, decreased waterfowl use (advantageous near certain facilities such as airports), and improved accessibility for upkeep (no standing water). Subsurface wetlands are also better suited for cold weather climates since they are more insulated by the earth (ITRCWT, 2003). Subsurface flow wetlands have the potential to clog and are therefore generally not as well suited to treating stormwater runoff due to sediment loads. The focus of the remainder of this section is on constructed surface flow wetlands for treatment of stormwater runoff.

Figure 5-24. Example Constructed Wetlands Used for Stormwater Treatment.

5.5.1.1 Site Suitability

Site characteristics determine the appropriateness of a treatment wetland for a particular location and influence the type of plants appropriate for the site.

Land area: The characteristics of the tributary area (size, imperviousness, and pollutants) to receive treatment and the treatment goals affect the area a treatment wetland may require, which can be considerable. A small wetland may have difficulty handling large flows from storm events in areas of high rainfall.

Topography: Site topography should include a suitable low lying area for the treatment wetland, and the site would ideally allow for gravitational flow into and out of the wetland; otherwise a pump station could be required.

Soil characteristics: Soil texture, compaction, hydraulic conductivity, pH, nutrient levels, minerals, salinity and toxicity affect plant establishment and growth. Clay soils with limited infiltration or the placement of a liner is necessary to create the saturated conditions for wetland establishment.

Groundwater: Wastewater infiltration to groundwater could present regulatory concerns and a high water table could make construction difficult.

Climate: Wetlands built in colder climates may require greater areas because treatment effectiveness is generally lower in colder weather. A consistent source of water is necessary to maintain wetland conditions during the dry season. Flows from urban areas (e.g., vehicle washing, excess irrigation) can help sustain wetlands during the dry season, but supplemental sources of water may be required

Pollutant concentrations: Stormwater runoff should not contain types of pollutants or pollutant concentrations that will impair wetland function; residual herbicides are an obvious example. Stormwater pollutant concentrations are generally amenable to wetland treatment; however, caution should be exercised as to the types of land uses receiving treatment, e.g., industrial areas.

5.5.1.2 Design Considerations

Surface treatment wetlands should provide for the retention of stormwater flows in the permanent water quality pool. Additional treatment capacity for stormwater can be incorporated into the wetland through the design of detention storage capacity above the permanent pool to provide treatment to a larger fraction of the stormwater runoff. If included, this additional storage volume is usually designed to drain in 24 hours. Whether or not additional storage capacity above the permanent pool is incorporated into the principal design, considerations for wet pond/constructed wetlands include:

Wetland vegetation: Vegetation suitable for wetlands should be selected for the pollutant uptake capacity and habitat characteristics. The preferred vegetation may not have desirable habitat characteristics, because wildlife such as birds can act as a source of bacteria.

Size: The design volume of the wetland basin should be sufficient to store the stormwater runoff volume anticipated from a design storm that will result in around 80% capture of the average annual stormwater runoff volume. A larger design volume will result in a larger fraction of the stormwater runoff captured by the basin, but will in turn require higher base flows to support the wetland flora. If sufficient base flows are not anticipated to support a large water quality pool, extended detention above the wetland should be considered to achieve treatment of a larger fraction of stormwater runoff.

Pool Characteristics: The treatment wetland water quality pool should have areas with a range of water depths to provide habitat for a variety of wetland plant species as well as containing open water areas over about half of the pool surface. The mix of depths and vegetation will support the various pollutant removal mechanisms that occur in a wet pond/wetland system. The pool should be designed to prevent short circuiting of flows through the wetland.

Presence of a Forebay: A forebay is the area where stormwater runoff enters the wetland that is designed to promote sedimentation of larger particulates (Section 5.3.4). The presence of a forebay may help improve sedimentation rates through settling of larger particulates in the forebay and finer particulates in the main water quality pool of the wetland. The main advantage of a forebay is in reduced maintenance cost by collecting sediment in one location that often has a hard bottom surface to facilitate sediment removal.

Additional design guidelines are given by ITRCWT (2003) and Kadlec and Knight (1996).

5.5.1.3 Pollutant Removal Mechanisms

Treatment wetlands are effective for treating a variety of pollutants. The mechanisms by which pollutants are removed are different for individual pollutants, depending on pollutant characteristics and local conditions (Kadlec and Knight, 1996). Wetlands provide a diverse array of unit processes ranging from size separation through sedimentation to nutrient uptake and microbially mediated transformations. Removal mechanisms that occur within treatment wetlands are summarized in Table 5-5.

Table 5-5. Pollutant Removal Mechanisms in Water Quality Treatment Wetlands

Pollutant	Removal mechanisms
Suspend solids	Sedimentation and filtration
Nitrogen	Nitrification and denitrification Plant uptake Volatilization
Phosphorus	Sedimentation Plant uptake Adsorption
Pathogens	Sedimentation and filtration Natural die-off / predation UV radiation
Toxic chemicals	Adsorption Degradation by bacteria Volatilization
Dissolved metals	Precipitation Adsorption

Sources: Strecker et al. [1992]; Brix [1993], Kadlec and Knight [1996], USEPA 2000c], IWA [2000],

5.5.1.4 Routine Operation and Maintenance Activities

Site Inspections: Inspections should occur on a regular basis to ensure proper operation, record observations, and initiate any additional operation or maintenance activities.

Water Quality Testing: Water quality testing should be conducted on a regular basis to evaluate the effectiveness of the treatment wetland. This information is necessary to assess if the wetland is operating as intended or if additional modifications may improve the pollutant removal effectiveness.

Trash and Debris Removal: Litter may be picked up at any time during site visits. Regular trash/debris removal will be performed at all sites on a quarterly basis and/or after storm events that result in heavy trash accumulations.

Vegetation Maintenance: Vegetation removal or thinning should be scheduled to minimize impacts to wildlife. Vegetation growth at inlets and outlets and along the perimeter should be inspected and removed or thinned as necessary.

Sediment Removal: It is recommended that a treatment wetland be designed with a forebay, open water area, or other sediment trapping area just downstream of its inlets. When operating properly sedimentation areas will capture the bulk of the sediment load in an easily maintained area and reduce the rate at which wetland fills in.

Pest / Invasive Species Management: Maintenance will be required to prevent or minimize the presence of unwanted pests at the treatment wetland. Mosquitoes are the most common problem and should be controlled through chemical or biological agents or species that prey upon mosquitoes (e.g., mosquito fish stocking, habitat enhancements for bats, larval controls). Vegetation that could provide a food source to rodents such as fruit and nut trees should be eliminated. Exotic or undesirable vegetation should be removed regularly. Local vector control agencies can provide the practitioner with information on the appropriate vector control measures for a particular area.

5.5.2 Flocculent / Precipitant Injection Systems

The purpose of flocculation is to form aggregates of small particles, a.k.a. (also known as) flocs, which can be removed by settling or filtration. Chemicals are added to the water receiving treatment to destabilize particles that promotes flocculation. Some common chemicals used in precipitation/coagulation/flocculation stormwater treatment include aluminum sulfate (alum), ferric chloride, ferrous sulfate, polymers, and lime. An innovative coagulant and adsorbent that has not been widely used for stormwater treatment is chitosan, a biopolymer of shrimp and crab shells. Similar to alum, chitosan causes coagulation of fine sediment particles, which then allows for gravity settling, biofiltration, sand filtration, or cartridge filtration. The Washington Department of Transportation used a chitosan for the treatment of construction site runoff from the Washington State I-90, Sunset Interchange Issaquah Project (WADOT, 2003). Chitosan was added to settling pond effluent, which caused the fine sediment particles to bind together enhancing removal during sand filtration. Chitosan may also remove phosphorus, heavy minerals, and oils from the water.

Common stormwater BMPs such as detention basins and retention ponds are limited in their effectiveness to remove fine particulates and associated constituents (e.g., metals and nutrients) from stormwater by gravity settling and physical flocculation processes. Due to small size and low settling velocities, only a small fraction of fine particulates is removed by gravity settling. The incorporation of chemical assisted precipitation/flocculation can provide effective treatment of fine particulates and associated constituents. Precipitation/flocculation treatment processes have also been shown to remove dissolved constituents (nutrients and metals), fats, and oil and greases from stormwater.

5.5.2.1 Pretreatment Considerations

Due to wide variability in stormwater characteristics, the precipitation process may require pre-treatment steps including flow equalization, oil removal, and neutralization. Flow equalization would prevent wide fluctuations in flow rate, temperature, and contaminant concentrations. Oil and grease solution may affect the settling of precipitates by creating emulsions. Provision of an oil-water separator, or skimming or coalescing would provide the necessary pre-treatment step.

5.5.2.2 Effectiveness Considerations

Precipitation/flocculation treatment involves three steps: coagulant mixing, flocculation, and gravity separation or filtration. Precipitation is the process of causing contaminants that are either dissolved or suspended in water to settle out of water as a solid precipitate (the floc), which then can be filtered, centrifuged or otherwise separated from the liquid portion. Precipitation is assisted through the use of a coagulant, an agent that causes smaller particles to be suspended in solution to agglomerate into larger aggregates. The performance of different chemicals for promoting flocculation varies depending on the characteristics of the stormwater to be treated and different chemicals will create varying amounts of sludge. In additional to chemical performance, consideration should be given to the cost of the chemical and the amount of sludge it generates (e.g., disposal cost). The optimal treatment strategy should be determined through jar testing based on recommended chemicals and dosages provided by chemical manufacturers.

5.5.2.3 Maintenance Considerations

Precipitation/flocculation systems require routine maintenance. These systems are usually automated chemical feed systems consisting of storage tanks, feed tanks, injection nozzles, overflow basins, mixers aging tanks, pipes, fittings, and valves. Regular maintenance activities include periodic flushing of the system, inspection and replacement of system parts (e.g., pump seals, pH electrode), testing and calibration, and sludge removal and disposal. Proper storage and handling instructions of chemicals along with Material Safety Data Sheets (MSDS) must be provided to the workers.

5.5.2.4 Other Considerations

The overall cost of chemical precipitation depends on many variables, including the characteristics of the influent quality, the chemical dosages to be used, the volume of water to be treated and the desired effluent quality. Moreover, chemical costs can vary widely depending on the form and quantity of material to be produced. Typically lime is least expensive of the common treatment options (USEPA, 2000b). As these are usually enclosed treatment systems, aesthetics are not a significant concern.

5.5.3 Aeration and Volatilization Devices

The processes of oxidation and volatilization may be enhanced by using sprinklers or aerators. Shallow treatment systems may not require mechanical aeration, as the surface area to volume ratio should allow sufficient wind-driven aeration. As treatment sites become deeper, oxygen depletion can become an issue. The principal aeration devices in water supply and wastewater systems may be classified as waterfall aerators, diffused-air (or pure oxygen) aeration systems, and mechanical aerators (Tchobanoglous and Schroeder, 1985). Examples of waterfall aerators include spray aerators, cascade aerators, and multiple-tray aerators. For moderately deep treatment systems, spray aerators such as surface splashers, sprinklers, and fountains, or diffused-air aeration systems that operate by introducing air by bubbles into the water column, can be used to enhance aeration. Paddle wheel aeration is also a common technique used for large ponds. Smaller fountains and paddle wheels may not sufficiently aerate the entire water column, due to the low volume of water that they draw. Larger fountains can potentially remedy this problem by using intake piping that draws water from near the bottom of the treatment system. Davis (1996) found a 50% reduction in BOD when surface splashing aeration was replaced with aspirating (deep injection) aeration. Similarly, underwater aeration systems can be placed along the bottom of very deep treatment systems such as wet ponds and permanent pools.

Typical designs for aeration systems in water supply and wastewater applications incorporate electric motors to drive the system. For stormwater applications, a more passive approach may be more desirable. This can be accomplished by enhancing aeration during transport to or away from the treatment site using waterfall aerators or baffles to cause turbulence in an inflow or outflow pipe or channel. A waterfall over a low-flow weir is another example of aeration taking place near the outfall of a site.

For sites where treating the volume of runoff is the main concern and water quality is generally not an issue, some methods of stormwater reuse can be quite effective at aerating water and volatilizing some constituents. Stored runoff can be conveyed through a sprinkler or irrigation system to water fields, yards, and gardens. By passing through the sprinkler system, the water is greatly aerated, enhancing its quality.

5.5.3.1 Effectiveness Considerations

Knowing the influent and effluent dissolved oxygen concentrations can help in the decision to incorporate aeration into a treatment site. A high dissolved oxygen (DO) concentration is one indicator of good water quality. Aeration also enhances water quality by 1) reducing or eliminating thermal stratification, 2) lowering iron levels through oxidation of Fe^{2+} to Fe^{3+}, which creates ferric phosphate precipitates (Zaw and Chiswell, 1999), 3) increasing the volatilization rate of volatile stormwater pollutants, such as ammonia, and 4) reducing algae growth due to reduced availability of nitrogen and phosphorus. For sites with high concentrations of influent nitrogen and phosphorus, aeration could also reduce effluent nutrient by increasing populations of aerobic bacteria, which require nutrients.

5.5.3.2 Maintenance Considerations

Aeration devices are generally self-sufficient once installed. Fountains and underwater aerators require electricity, and the power supplies need to be maintained. It may be possible to use solar or wind energy, and potentially even micro-hydroelectric turbines at stormwater basin outfalls to continuously power aeration devices. Structural modifications such as baffles will need to be inspected regularly to maintain functionality and clear clogs or impediments.

5.5.3.3 Other Considerations

Systems with high flow rates and water velocities probably do not require aeration, because high velocities should create sufficient aeration through mixing. If influent flow rates and velocities are low, aeration and mixing could provide hydraulic benefits by reducing temperature gradients and the propensity for thermal stratification-induced short circuiting. A baffle system that highly aerates the water will also reduce flow velocity; thus increasing the hydraulic retention time. Systems with long residence times and deep permanent pools will often need some sort of mixing or aeration keeps the systems aerobic.

5.5.4 Disinfection Systems

Disinfection systems use chemical or physical processes to kill microorganisms. A wide range of chemical compounds can be used, with chlorine and ozone the most common. Physical processes include the use of heat, sound, or light with light the most commonly used physical method for treatment due to cost. While other methods are available, such as boiling or alternate chemicals, only the most commonly used disinfection systems for stormwater applications, chlorine, ozone, and UV, are discussed further. Disinfection systems are most commonly used to treat water supplies, but may be appropriate for stormwater applications if pathogens are particularly problematic.

5.5.4.1 Recommended Pretreatment

The incidence of sediments and other stormwater constituents can reduce the effectiveness of the disinfection process. Some form of pretreatment such as sedimentation or filtration should be provided upstream of disinfection systems to mitigate solids and other stormwater constituents that interfere with disinfection process. The recommended pre-treatment options for disinfection include practices that mainly remove solids and organics from stormwater. The incidence of solids affects all three disinfection technologies particularly UV through shielding microorganisms. The presence of organics can also absorb light, reducing the effectiveness of UV treatment, and can result in undesirable byproducts when combined with ozone or chlorine treatment.

5.5.4.2 Effectiveness Considerations

The effectiveness of any disinfection operation depends on the technology, the influent quality and the effectiveness of maintenance practice (Table 5-6). Effective pretreatment and regular maintenance are the surest ways to enhance and maintain disinfection performance. Increasing the contact time between the disinfection agent and target organisms results in greater kill rates. For chlorine compounds and ozone disinfection, the concentration of the compounds used in the disinfection process is related to the die-off rate of the target organisms, while for UV disinfection the intensity of the UV radiation is similarly related to the die-off rate of the target organisms.

Disinfection facilities are usually more complex than most other tertiary treatment options. Disinfection facilities can be designed to accommodate a wide range of flow rates, but in general, higher flow rates require more costly equipment. To enhance treatment through flow control and minimize costs, disinfection operations may benefit from upstream peak attenuation BMPs that control flow rates, resulting in more consistent flows and requiring smaller disinfection facilities with reduced costs.

5-6. Effectiveness Considerations

Method	Enhance	Inhibit
Chlorine	Thorough initial mixing, sufficient contact time, microorganisms – bacteria more readily killed with chlorination,	Reacts with organics and other compounds, particularly sulfuric compounds creating unwanted byproducts and interference, particulates inhibit effectiveness, microorganisms – viruses not as easily killed with chlorination as bacteria, age of organisms – older organisms more resistant,
Ozone	High transfer efficiency of ozone to wastewater,	Chemical characteristics – organics and sulfuric compounds interfere, particulates inhibit effectiveness
UV	Output UV energy per flow volume: combination of intensity and exposure time,	Non-ideal hydraulics can result in inconsistent exposure, particulates inhibit effectiveness, light absorbing compounds such as dyes can inhibit effectiveness, buildup of films on lights inhibit effectiveness requiring regular cleaning,

5.5.4.3 Maintenance Considerations

Disinfection systems require regular maintenance in order to maintain consistent performance. Maintenance activities for chlorine disinfection plants typically include disassembling and cleaning the various components of the system, removing iron and manganese deposits, maintaining pumps, and testing and calibration of equipment (USEPA, 1999a). Typical maintenance activities for ozone disinfection include calibration of monitoring instrumentation, refurbishment of air preparation equipment and ozone generator, inspection and cleaning of the ozonator, air supply and dielectric assemblies. It is also important to constantly monitor and maintain ambient ozone levels below limits of applicable regulations (USEPA 1999b). The main maintenance activity for UV disinfection is cleaning the UV lamps. The quartz tubes on the UV lamps must be routinely cleaned to reverse the effects of fouling, which minimizes light penetration into the influent and can greatly impact the performance of the system. The formation of biofilms on the exposed surfaces of the UV reactor can negatively impact performance since biofilms can shield bacteria. Covered UV channels are less susceptible to biofilm formation. Occasionally cleaning channels with hypochlorite, paracetic acid or other suitable agents will remove biofilms (Metcalf and Eddy, 2003).

5.5.4.4 Other Considerations

Disinfection systems need specialized equipment that is usually housed in a room. The space requirements depend on the target flow rates and the type of equipment selected for the disinfection process. UV and ozone disinfection typically require less space than chlorine disinfection. As disinfection systems are typically housed indoors aesthetics will generally depend on the structure and are considered of low importance.

The cost of chlorine, ozone and UV disinfection alternatives vary according to site conditions, influent quality and the designs selected. However, ozone disinfection tends to be more expensive than the other alternatives (USEPA, 1999b). UV disinfection costs are comparable to chlorine disinfection costs if the cost of de-chlorination is added (USEPA, 1999c).

Improvements in technology continue to lower the cost of all three disinfection alternatives; however, the greatest reductions in costs are likely to be realized in UV disinfection.

Advantages and disadvantages of chlorine, ozone, and UV disinfection systems are summarized in Table 5-7.

Table 5-7. Advantages and Disadvantages of Disinfection Systems.

Method	Advantages	Disadvantages
Chlorine	Low cost, deodorizes, effective disinfectant, can leave a residue that can prevent reestablishment of microorganisms in water,	High toxicity to higher life forms, safety concerns for workers, very corrosive, reacts with organics and other compounds forming unwanted byproducts, dechlorination can be required to protect receiving waters, requires relatively long contact time compared to other methods,
Ozone	Deodorizes, effective disinfectant, more effective viricide than chlorine, increased dissolved oxygen concentration, no chemical residue as with chlorine	Reacts with organics and other compounds forming unwanted byproducts and creating ozone demand, toxic to higher life forms, corrosive, energy intensive adding to cost, no residual effect
UV	Effective disinfectant , no residual toxicity, more effective viricide than chlorine, little to no formation of unwanted byproducts, generally safer than chemical disinfectants	Energy intensive adding to cost, no residual effect, hydraulic design is critical to performance, no odor control

5.6 Hydrologic/Hydraulic Controls

Hydrologic / Hydraulic (H/H) controls refer to the components that are applied after flows have been concentrated in the drainage system, roadside ditch, or treatment BMP. The level of treatment attainable in some BMPs is dependent on the characteristics of the flow routed to, through, and from them; therefore, inlet and outlet structures and other flow controls may greatly influence their performance. For instance, outlet structures are used to control many of the flow characteristics within BMPs including detention time, short-circuiting, sedimentation and the flow characteristics of the effluent. However, some H/H controls are not explicitly included for water quality treatment but rather function more in a flood or erosion control capacity. Examples of H/H controls include flow splitters, energy dissipators, check dams, berms, weirs, orifices, perforated risers and other multi-stage outlet designs, infiltration and exfiltration trenches, evaporation enhancement systems, and many other flow control and volume reduction devices. Due to the many different types and configurations of H/H controls, only a brief introduction is provided herein with a discussion of the applications of the controls and/or methods to enhance the use of the controls. References to guidance manuals and online sources of additional information are provided in an effort to avoid duplication of information.

5.6.1 Flow Splitters

There are many different configurations of flow splitters, but they all are generally based on two different design methods: storage and flow restriction (Metro Council, 2001). The storage method is based on a detention unit that simply bypasses flows (or overflows) via a weir or standpipe or other outlet device (see next subsections) once the design volume (or stored depth) is exceeded. The flow restriction method is based on a flow-stage proportioned device designed to overflow once the design flow rate (or flow depth) is exceeded.

Flow splitters may be placed anywhere in a treatment system, but are most commonly used to divert stormwater runoff from a storm drain or channel to a BMP or to allow bypass and prevent flooding when the capacity of the BMP is exceeded. If flows are being diverted from an existing storm drain, the potential for upstream backwater effects should be evaluated. Figure 5-25 illustrates a generic flow splitter device used to divert a water quality design volume or flow rate to a BMP.

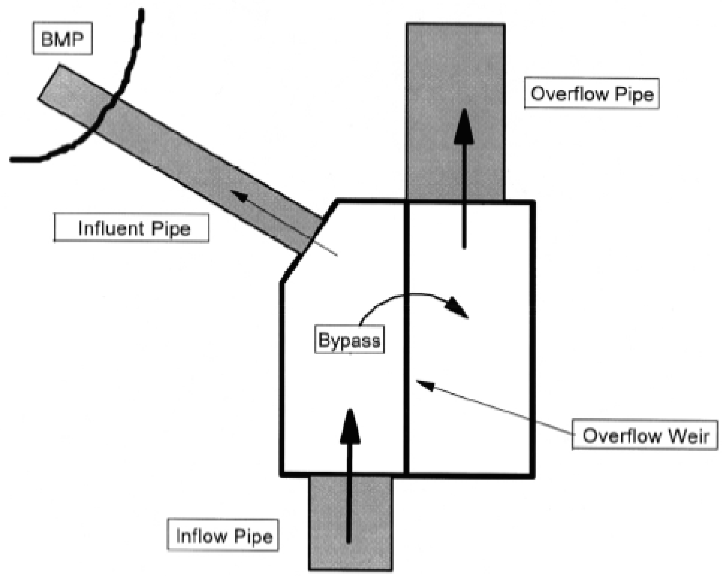


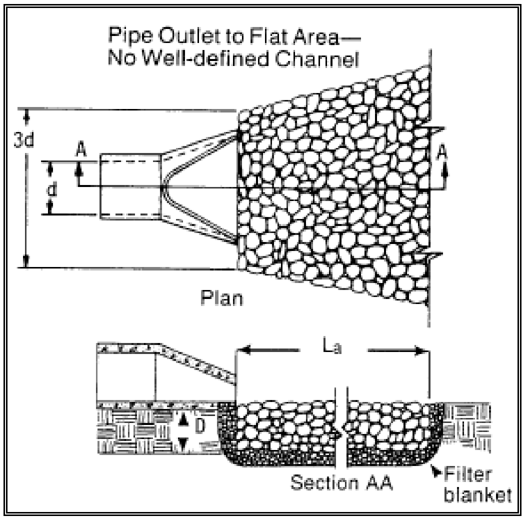
Figure 5-25. Schematic of a Generic Flow Splitter.
Source: Metropolitan Council (2001).

5.6.2 Energy Dissipators

Energy dissipators are engineered devices placed at BMP inlets and drainage system outlets to minimize erosion and scour due to high impact flows. Man made conveyance systems typically deliver flows at design velocities that far exceed the velocities at which erosion starts to become a problem at the receiving water body (or BMP entrance). Energy dissipators allow conveyance systems to operate at high velocities without causing erosion problems at the receiving water body by serving as flow transition structures that absorb the initial impact and reduce velocities to less erosive levels (ARC, 2001).

Energy dissipator designs generally use rock, riprap or concrete as materials for absorbing the impact on the incoming high energy flows. Some examples of energy dissipator designs are riprap aprons (Figure 5-26), riprap outlet basins and baffled outlets (ARC, 2001). A detailed design of energy dissipators has been omitted in this discussion. The reader is referred to the Federal Highway Administration Hydraulic Engineering Circular No. 14 entitled, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, for detailed design guidelines on energy dissipators. However, some of the factors to be considered in the selection and design of energy dissipation structures include (FHWA, 1983):

- The design discharge
- Outlet flow conditions (velocity and depth)
- Outlet slope
- Operation characteristics and performance curve
- Standard culvert outlet design utilized: projection, wingwalls, headwalls and aprons



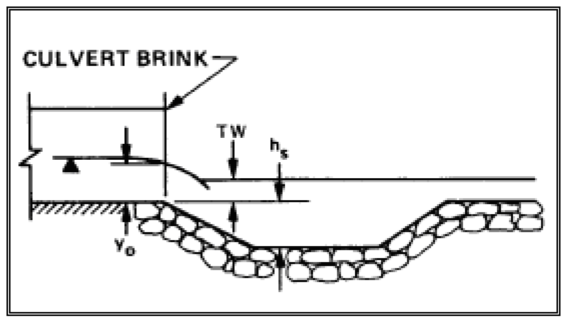


Figure 5-26. Typical design of a rip rap apron at a stormwater outfall.

Source: Georgia Stormwater Manual (http://www.georgiastormwater.com/vol2/4-5.pdf)

Well designed and constructed energy dissipators are durable structures requiring little maintenance. For early detection of structural problems and other maintenance needs, routine inspections and inspections after large events are recommended. Energy dissipators provide protection against scour and erosion at a low cost relative to other treatment system components with minimal impact to aesthetics.

5.6.3 Check Dams

Check dams (Figure 5-27) are small barriers constructed of rock, concrete, wood, gravel bags, sandbags, fiber rolls, straw bales, or reusable products, placed across the drainage paths of stormwater controls or used as erosion control devices (CSQTF, 1993) Check dams are usually considered in the context of erosion control; however, check dams can be used to enhance open channel BMPs such as biofilters. Check dams have also found application in the field of stream restoration, which is beyond the scope of this document. Check dams can reduce flow velocities in vegetated swales by serving as obstructions, with a consequent reduction in flow energy and increase in sedimentation processes. The check dam itself serves as a barrier to trapped sediment and other solids that may otherwise be resuspended during high flows.

To ensure consistent performance, check dams should be routinely inspected and the accumulated sediment should be removed. Inspections after large storm events are recommended to detect structural damage that can lead to check dam failure and release of previously captured sediment into the drainage system. Check dams are best used as enhancements to other BMPs except in erosion control applications where they are effective as stand-alone BMPs.

Figure 5-27. Roadside swale with wood check dams.

5.6.4 Berms

Berms are embankments (often built with earthen material) that provide surface containment of stormwater, either by partitioning a larger impoundment (Figure 5-28) or as part of the embankment for the entire impoundment. Volume-based BMPs often employ the use of earthen berms because other materials such as plastic or concrete walls are expensive. BMPs that employ or benefit from berms include ponds, biofilters, infiltration basins, and media filters. Berms provide storage volume for water quality treatment flows in most cases and occasionally flood control storage volumes. Berms can be built with sandbags, earth, and possibly even construction waste materials. BMPs that are dependent on the flow path length through the BMP can benefit from internal berms positioned to increase the flow path (and therefore the hydraulic retention time) in the BMP.

Structural (hydraulic, geotechnical) considerations for embankments may be found in UDFCD (1999) and other publications, such as Bureau of Reclamation (1987). Such related civil engineering considerations are as important as the depth of storage behind the embankment and/or the consequences of downstream flooding due to failure.

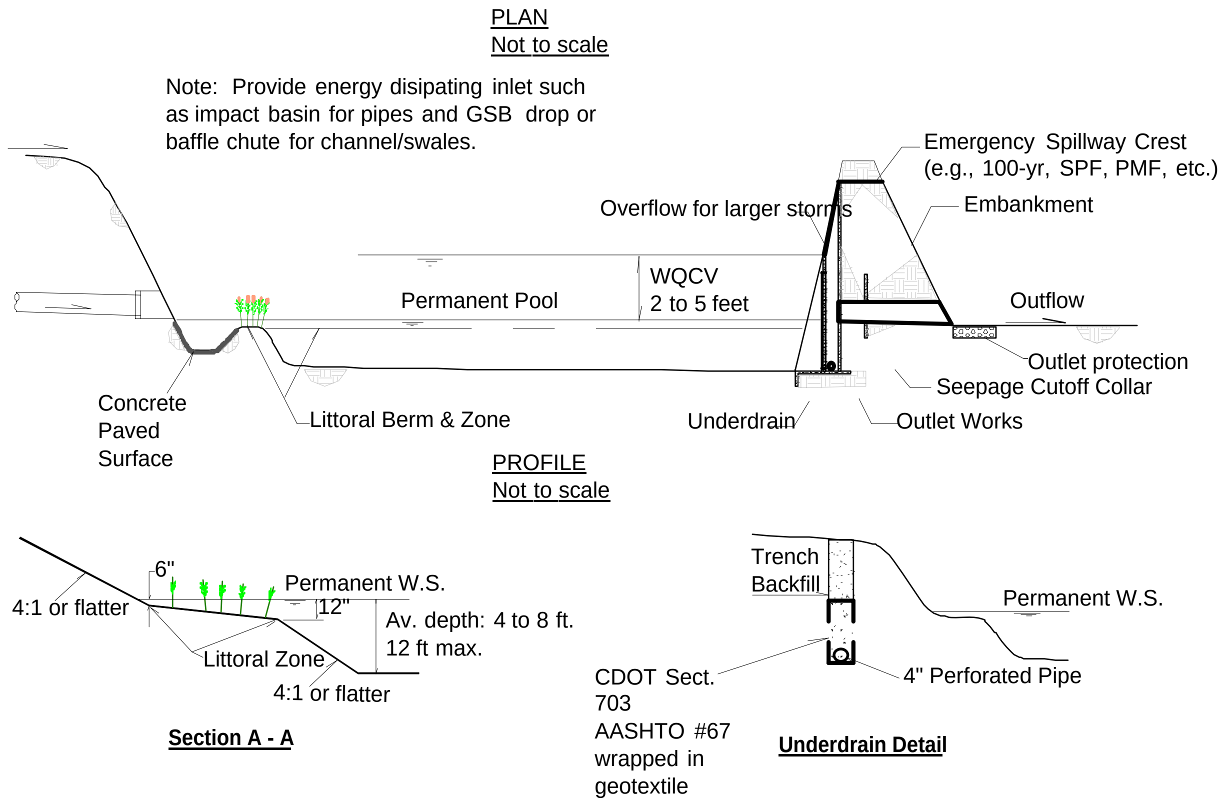


Figure 5-28. Berm as part of a retention pond.

Source: Urban Drainage and Flood Control District (UDFCD, 1999).

Small berms require little maintenance; however, periodic inspections are recommended to ensure that berms remain structurally sound. Due to the potential flooding that may result from berm breaches, inspections should be conducted after every large event. Vegetated berms may need to be mowed and occasionally reseeded to maintain vegetation. For ease of maintenance and safety when traversing, larger embankment side slopes should not be steeper than 4 horizontal to 1 vertical (WEF and ASCE, 1992). Ponds greater than 4-5 feet may require special design approval from the local flood control agency.

5.6.5 Weirs

A weir typically consists of an obstruction placed across the drainage path in an open channel or within a closed conduit operating under open channel flow-type conditions such that water flows over the weir's edge or through well defined openings in the weir. There are many kinds of weirs with various discharge properties. Weirs commonly used for flow control include the sharp crested weir, the broad crested weir and the V-notch weir. Weirs commonly used for flow measurement include the rectangular weir, the trapezoidal weir and the triangular weir (ASCE/EPA, 2002). There is a specific discharge equation associated with each weir, a few of which are presented as Equations 5-9 through 5-14. A brief description of some of the commonly used flow control weirs is presented in the subsequent sections below.

5.6.5.1 Sharp-Crested Weirs

Sharp crested weirs (Figure 5-29) are weirs that have a thin leading edge such that the water springs clear as it overflows (ARC, 2001). A modification of sharp crested weirs is the end-contracted sharp crested-weir so named because this class of weir causes through flows to contract. Sharp crested weirs are often used as measurement devices because they have stable stage-discharge relations. The discharge through a sharp-crested weir with no end contraction can be modeled by Equation 5-9.

$$Q = [(3.27 + 0.4(H/H_c)]LH^{1.5} \text{ (ARC, 2001)} \qquad [5\text{-}9]$$

Where: Q = discharge (cfs)
H = head above weir crest excluding velocity head (ft)
H_c = height of weir crest above channel bottom (ft)
L = horizontal weir length, perpendicular to flow (ft)

The discharge from a sharp crested weir is reduced by submergence when the tail water rises above the weir crest. The discharge equation for a submerged sharp crested weir is presented in Equation 5-10.

$$Q_s = Q_f(1-(H_2/H_1)^{1.5})^{0.385} \text{ (ARC, 2001)} \qquad [5\text{-}10]$$

Where: Q_s = submergence flow (cfs)
Q_f = free flow (cfs)
H_1 = Upstream head above crest (ft)
H_2 = Downstream head above crest (ft)

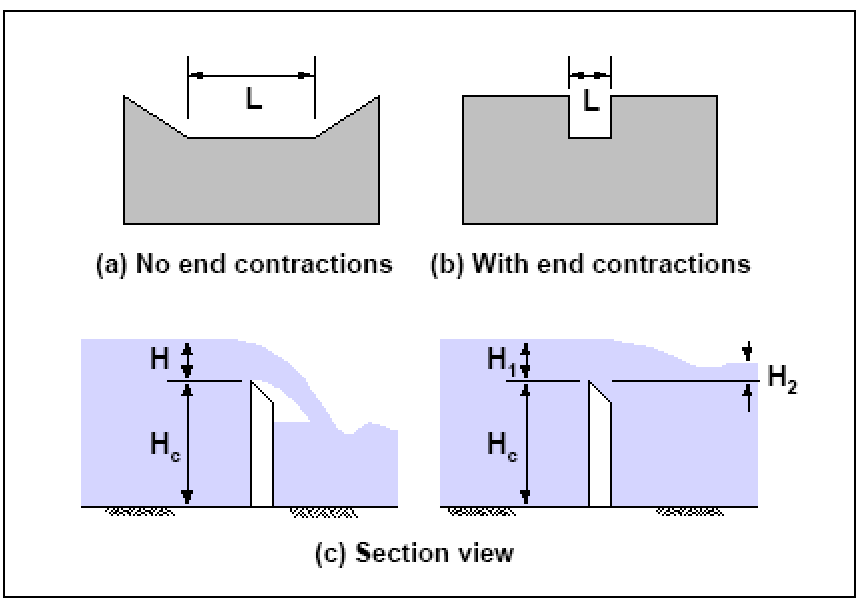


Figure 5-29. Schematic of a sharp-crested weir.

5.6.5.2 Broad-Crested Weirs

A broad-crested weir (Figure 5-30) is a weir with a relatively long raised channel control crest section. True broad-crested weir flow occurs when the upstream head above the crest is in the range of about 1/20 to ½ the crest length in the direction of flow (ARC, 2001). The discharge equation for a broad-crested weir is presented in Equation 5-11.

$$Q = CLH^{1.5} \text{ (ARC, 2001)} \qquad [5\text{-}11]$$

Where: Q = discharge (cfs)
H = head above weir crest (ft)
C = broad-crested weir coefficient
L = broad-crested weir length perpendicular to flow (ft)

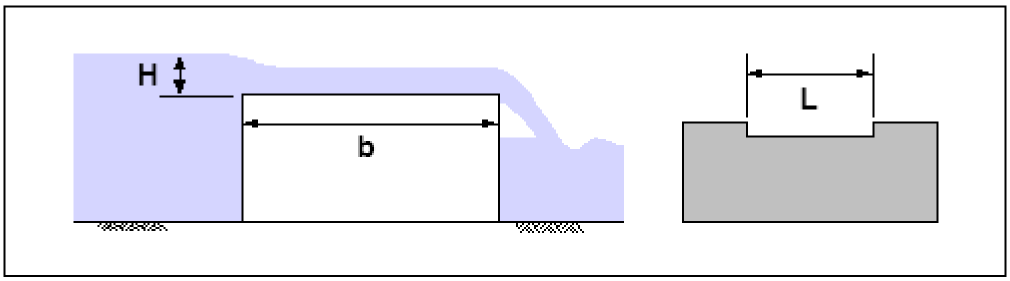


Figure 5-30. Schematic of a broad-crested weir.

Values of C (U.S. customary units) range from a maximum of about 3.087 (for a broad-crested weir with a rounded upstream edge with a slope as large as the headloss due to friction that prevents contraction) to a minimum of about 2.6 (for a broad-crested weir with sharp corners).

5.6.5.3 V-Notch Weirs

A V-notched weir is a weir with a V-shaped notch as shown in Figure 5-31. The discharge equation for a V-notch weir is presented in Equation 5-12.

$$Q = 2.5\tan(\theta/2)H^{2.5} \text{ (ARC, 2001)} \qquad [5\text{-}12]$$

Where: Q = discharge (cfs)
H = head on apex of V-notch (ft)
θ = angle of V-notch (degrees)

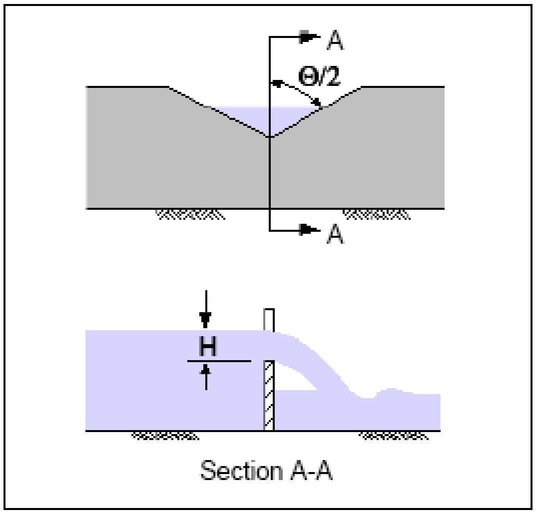


Figure 5-31. Schematic of v-notch weir

5.6.5.4 Proportional Weirs

A proportional weir (Figure 5-32) is somewhat more complex to design and construct than the other weirs discussed previously. However proportional weirs have the added advantage of a linear discharge relationship that is achieved by varying the discharge area nonlinearly with head. Equation 5-13 presents the discharge relation for a proportional weir and Equation 5-14 can be used to evaluate the dimensions of a proportional weir.

$$Q = 4.97a^{0.5}b(H - a/3) \text{ (ARC, 2001)} \qquad [5\text{-}13]$$

$$x/b = 1 - (1/3.17)(\arctan(y/a)^{0.5}) \text{ (ARC, 2001)} \qquad [5\text{-}14]$$

Where: Q = discharge (cfs)
a, b, H, x, and y are the weir dimensions shown in Figure 5-32, in U.S. customary units (ft).

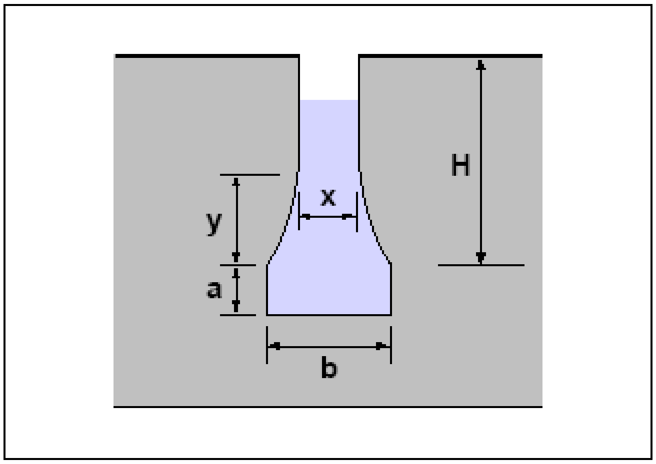


Figure 5-32. Schematic of a proportional weir.

In general weirs are low cost, easy to install and can be quite accurate flow measurement devices when used correctly. Weirs can be used to regulate flows in natural channels with irregular geometry providing more reliable estimates than would have been achieved with Manning's Equation. Weirs modify the hydraulics of the existing system by introducing obstacles in the flow path; therefore the engineer must ensure that the design capacity of the drainage element with the weir will be adequate to avoid flooding of upstream structures. Sediment and debris that accumulate behind a weir can alter the hydraulic conditions, changing the empirical relationships between flow depth and discharge rate. Therefore weirs should be inspected regularly and accumulated sediment or debris removed. For flow measurement applications, if high sediment loads are anticipated, a flume may be a better option (ASCE/EPA, 2002). Weirs can be used as inlet/outlet and/or flow control or flow measurement structures in ponds, biofilters, wetlands and open channel systems.

5.6.6 Orifices

Orifices are flow control devices used either within a flow splitter or as part of an outlet design of a detention facility. For water quality outlet designs, the use of a single orifice is not recommended because the stage-storage discharge relationship cannot be optimized to achieve the maximum hydraulic retention time for a variety of flow rates. Instead multiple orifices configured in a perforated riser or at different stages on an orifice plate are recommended (see next section).

The orifice equation is presented in Equation 5-15

$$Q = CA\sqrt{2g(H_1 - H_2)} \qquad [5\text{-}15]$$

Where: Q = flow rate (cfs)
C = orifice discharge coefficient
A = cross-sectional area of orifice (ft^2)
H_1 = Upstream head above centroid of orifice opening (ft)
H_2 = Downstream head above centroid of orifice opening (ft)
g = gravitational constant (32.2 ft/s^2)

If the orifice outlet is submerged then H_2 is nonzero, but if the outlet is freely discharging to the atmosphere H_2 is approximately zero and only the upstream head is considered. The orifice coefficient, C, ranges from about 0.6 for sharp-edged orifices to about 0.9 for well-rounded orifices. Refer to any fluid mechanics text or hydraulic design manual for more information on orifice designs and related coefficients.

As a general rule, orifices should not be smaller than about 2-inches in diameter to avoid clogging unless adequate screening of trash and debris is provided in front of the orifice.

5.6.7 Multistage Outlet Designs

While weirs and orifices are important stand alone flow control and measurement design elements they often need to be coupled in an integrated outlet design to achieve desired water quality and flood control protection. Outlet controls may include complex weir designs, multiple orifices, perforated riser pipes, and spillways with the goal of controlling and releasing both large storms and water quality storms. Orifices and risers are outlet structures that can be designed to provide a wide range of discharge rates over a wide range of depths. Multiple orifices are preferred over single orifices for water quality because both small and average sized storms can be captured and detained for adequate retention times (i.e., 24-72 hours). A perforated riser is a standpipe with perforations at specific locations along its length and often includes an orifice plate at the bottom of the riser (ARC, 2001). Equation 5-16 can be used to estimate the flow through a perforated riser as illustrated in Figure 5-33. Note that this assumes all of the perforations are of the same size.

$$Q = C_P \frac{2A_P}{2H_S} \sqrt{2g} H^{3/2} \qquad [5\text{-}16]$$

Where: Q = flow rate (cfs)
C_p = discharge coefficient for perforations (normally 0.61)
A_p = sum of cross-sectional areas of all perforations (ft^2)
H_s = distance from S/2 below the lowest row of holes to S/2 above the top row (ft)

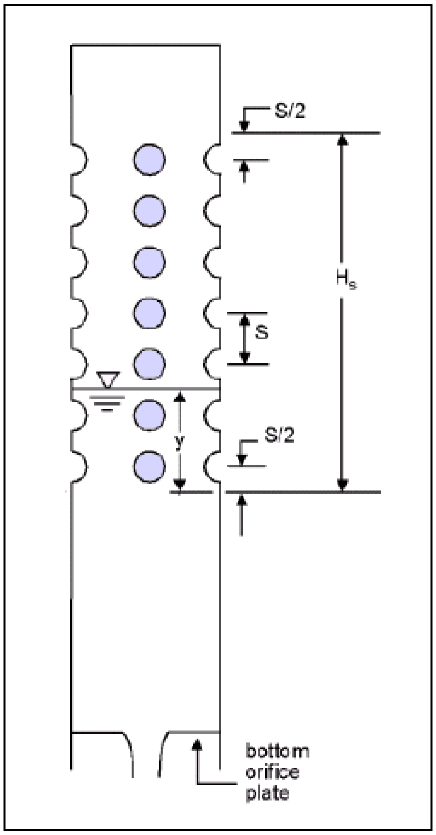


Figure 5-33. Schematic of a perforated riser.

(UDFCD, 2001)

Figure 5-34 is an example outlet structure design that includes a perforated riser structure for water quality control, two large orifices, and one emergency spillway. Notice that in this design a gravel jacket is used to conceal the riser and prevent clogging of the perforations. Figure 5-35 is an example of a two-stage riser outlet with a simple trash rack used to prevent clogging. Additional information on multi-stage outlet design may be found in the Georgia Stormwater Management Manual, Volume 2 (ARC, 2001), the Urban Drainage Storm Criteria Manual (UDFCD, 1999), or the Stormwater Management Manual for Western Washington, Volume III (WADOE, 2001). Section 7.6.2 also provides a more detailed discussion of how different multistage outlet controls may be used to improve hydraulic retention in stormwater detention facilities.

Figure 5-34. Example of multiple stage outlet design structure for a retention pond.

Figure 5-35. Example multiple stage spillway risers with trash rack. (USEPA, 2004)

5.6.8 Infiltration/Exfiltration Trenches and Basins

Infiltration/exfiltration trenches and basins utilize infiltration reduce or eliminate stormwater runoff, as previously discussed in Section 5.4.3. Stored stormwater infiltrates into the surrounding soil at a rate dependant upon the hydraulic properties of the local or engineered soil and upon depth to groundwater, which affects ponding. It has been recommended that the groundwater table not be less than 0.5 m below bottom of the infiltration basin (Novotny, 1995). Each basin requires excavation of soil to obtain the required stormwater runoff design volume. Excavations are generally backfilled with gravel aggregate to alleviate scour effects and aid infiltration through the soil (Mays, 2001). Infiltration trenches can be installed in a highly localized environment, such as around the perimeter of buildings, if soil conditions are appropriate and adequate moisture barriers are used alleviate structural concerns. The influence of disturbed urban soils on infiltration rates must be considered (see Section 6.7, Soil Properties).

5.6.9 Evaporation Enhancement Systems

These systems, or BMP components, include the use of open water ponds, sprinklers, fountains, and vegetated areas to enhance volume loss due to evaporation and evapotranspiration (ET). In order to promote evaporation from a water surface or ET from a soil-vegetation matrix, detention must occur. The effectiveness of ET depends on several meteorological factors, such as wind speed, cloud cover, and humidity as well as other factors such as soil and vegetation type. A good example of a BMP that primarily uses these integrated unit processes is a green roof, where soil and vegetation are placed on the impervious surface of a roof to provide peak flow attenuation, as well as reduce the total volume of runoff through evapotranspiration. While there would be some filtration and adsorption of atmospheric pollutants, concentrations would typically be low compared to runoff that has made contact with impervious surfaces such as roads.

Except for extremely arid environments, evaporation enhancement may only provide a small reduction in volume as compared to infiltration. The use of evaporation enhancement systems that require energy input, such as sprinklers or fountains, may only be practicable if volume loss is highly desirable or other benefits such as volatilization or aeration are needed to improve BMP performance. See Sections 4.2.4, 4.2.5, and 5.5.3 for a discussion of these other processes and the potential devices that can be used to enhance them.

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CHAPTER 6 PRACTICABILITY ASSESSMENT OF CANDIDATE BMP SYSTEMS

Determining the practicability of candidate BMPs for a site is not a decision based specifically on performance or hydrologic measures, but rather on feasibility of design, installation, and implementation. The purpose of the practicability assessment is to limit the candidate BMP designs to the most viable alternative(s) for preliminary sizing and screening-level assessment. There are several factors that engineers must take into account when initially selecting a particular BMP design to pursue, including performance for target pollutants, hydrology and hydraulics, surface and subsurface space availability, maintenance, costs, and aesthetics. Other factors that must be considered include safety and human health concerns, regional constraints, downstream impacts, regional and climatic concerns, and overall project budget (cost considerations). As many practicability factors are highly site-specific, all potentially relevant factors could not be included in this document. The design engineer should use the information presented in previous chapters to account for site-specific conditions as much as possible, but additional sources of information may also be required. The following subsections present guidance for reducing the number of potential BMP alternatives based on common practicability factors. Worksheets 2 and 3 in Appendix D summarize these and several other practicability factors for both standard structural BMPs and proprietary BMPs, and can be used to quickly evaluate BMP/LID alternatives.

6.1 Space Availability

A key consideration for BMP/LID selection and design is the amount of available space. For the highway environment, space availability may vary significantly, but typical open space areas include roadside embankments, medians, cloverleaves, and near on-ramps and off-ramps. In urban areas, highway right-of-way (ROW) may be limited due to previous widening projects and build-out of surrounding land uses, while in rural areas there are often large open spaces adjacent to and in the medians of the primary travel lanes. However, the existence of open space does not necessarily constitute the availability of space for a structural BMP since planning for future expansions may take precedence over stormwater control projects. Thus, the only feasible option for highly urbanized or urbanizing areas may be subsurface devices that can be located within the storm drain system, such as underground tanks/vaults, hydrodynamic devices, media filters, and catch basin inserts. However, these types of devices generally provide a lower level of treatment and are more difficult to monitor and maintain than surface BMPs.

Detention-type BMPs usually require a larger footprint than flow through-type BMPs, but the amount of space required for a particular BMP is directly dependent upon the amount of runoff the BMP is expected to treat and the desired hydraulic retention time (HRT) for flows through the BMP. Since the runoff volume is directly proportional to the drainage area and the percent imperviousness, the size of the facility is a function of the size and land use characteristics of the catchment. The linear, directly connected impervious nature of the highway environment often restricts the size of the catchment that can be reasonably treated with a single BMP. Therefore, rather than large regional facilities located near the downstream end of a watershed, a more distributed, LID approach to stormwater management is often more feasible. With the establishment of municipal watershed partnerships, it may be possible to treat highway runoff in regional, offsite BMPs with the benefit of centralizing maintenance activities.

6.2 Existing Infrastructure

One of the primary drivers for selecting and sizing a BMP for a site is the existing infrastructure. A retrofit to an existing drainage system is more restrictive than the construction of a new one. Also, the design of roadways and bridges and the presence of utilities may inhibit BMP selection and placement. Furthermore, concern over the structural integrity of roadways, shoulders, footings, bridge abutments, and retaining walls may discourage certain roadside infiltration/exfiltration practices.

6.3 Hydraulic Gradient and Slope

The available hydraulic gradient at a site is another factor that must be considered when selecting and designing a BMP or LID facility. A slope that is too mild may cause ponding and backwater effects, which in turn may cause premature sedimentation and clogging of inlet pipes. A slope that is too large may cause scour at the inlets and outlets of a facility. While some designs may be modified to accommodate larger slopes, such as using check dams and energy dissipators, many BMPs will not function properly or may cause slope failure if slopes are too great. For instance, unlined ponds should never be placed on hillsides, and a slope stability analysis should be conducted for any potential BMP location where slopes are greater than about 15%. Also, many types of BMPs require sufficient hydraulic head for proper operation. For example, swales must have sufficient longitudinal slope to avoid ponding. Furthermore, some inlet devices require a minimum amount of space between the inlet and storm drain invert in order to fit in the catch basin. The ability to design a BMP treatment train is also dependent on the available hydraulic gradient between the inlet and final discharge point.

6.4 Public Acceptance (Aesthetics, Property Values)

Public acceptance could be measured by market and preference surveys, reported nuisance problems, visual aesthetics and potential impact a BMP would have on the neighborhood property values. Some BMPs, such as biofilters, are unobtrusive and in general tend to look more natural and are more easily disguised than non-vegetated practices. In contrast, dry ponds and infiltration basins that accumulate trash and debris and sediment loads provide negative aesthetic impact as compared to other treatment practices. Odors, mosquitoes, weeds, and litter can all be potential problems in stormwater BMPs. However, the negative aesthetic impacts could be nullified through regular maintenance to remove debris and a good landscaping plan.

6.5 Property Ownership

The ownership of property may also have a significant effect on the selection and design of a BMP or LID facility. The most strategically located parcel for a BMP may be privately owned or owned by a public agency other than the DOT implementing the BMP. In these instances the property must be purchased or an access agreement established. Implementation costs can be significantly increased if property must be purchased. Also, some agencies may be resistant to certain BMPs on their property. Internal conflicts on property ownership may also be a concern. For instance, the maintenance department may own land that the watershed management department would like to use for a BMP project. While this is generally an easier agreement to establish, there may be conflicting planned uses for the property that must be negotiated.

6.6 Health and Safety

BMPs potentially could create a public health hazard by increasing habitat availability for aquatic stages of mosquitoes, and by creating harborage, food, and moisture for other reservoir and nuisance species. Emerging public health threats, such as the detection in 2001 of the exotic Asian tiger mosquito and the westward expansion of mosquito-borne West Nile virus illustrate the importance of cooperation and partnership at all levels of government. The public health powers of state departments of health and safety (DHS) including the power to abate public nuisances and those of local mosquito and vector control agencies must be considered. For example, eight mosquito species have been collected from Caltrans BMP structures, four of which are vectors of human disease (Metzer, 2001). Of the eight different BMP technologies implemented by Caltrans, those that maintained permanent sources of standing water in sumps or basins provided excellent habitat for mosquitoes and frequently supported large populations relative to other designs. In contrast, BMPs designed to drain rapidly provided less suitable habitats and rarely harbored mosquitoes. Information with regards to what factors within BMPs are most conducive to mosquito production and which species utilize these structures should be considered. Based on these findings, appropriate engineering modifications should be made to minimize the potential of certain BMPs to produce or harbor vectors.

In addition to vectors, it is also critical that stormwater treatment structures do not create public health hazards. Detention BMPs, depending upon their depth, could pose safety hazards, particularly for children, if they are not properly signed and fenced.

6.7 Soil Properties

In addition to the slope, the type of soils and geologic formations at a site may dictate the type and design of a stormwater BMP. Soils that are highly erosive or cut slopes that contain a slip plane that is prone to failure should be avoided. BMPs and LID facilities that rely on infiltration must have well-drained underlying soils, and the depth to bedrock must be sufficient not to cause excessive ponding. For BMPs designed to have a permanent pool, soils classified as NRCS hydrologic soil groups C or D may be desirable. If native soils are in soil groups A or B, a clay or geotextile liner would likely be needed to maintain the permanent pool. Soil is an integral part of the hydrologic cycle, as it regulates the processes of surface runoff, infiltration and percolation, and is a major controlling factor in evapotranspiration through the capacity of the soil to store and release water.

Soil characteristics are extremely variable, even for locations just a few meters apart. Moreover, in urban areas, disturbed (often compacted) soils bear little resemblance in their physical properties to their natural state (Pitt et al., 2001). The importance of local, site-specific measurements of infiltration cannot be overemphasized.

6.8 BMP Performance Evaluation

Estimating the treatment performance of a BMP requires an evaluation of 1) runoff volume reductions, 2) capture efficiency, and 3) expected effluent quality for target constituents.

6.8.1 Volume Reduction

There is certainly a basis for factoring in volume and resulting pollutant load reductions into performance estimates, particularly for the case of TMDLs. The infiltration capacity of the soils in a BMP primarily influences the volume reduction through infiltration to subsurface, and, combined with vegetation, ET. Soils with a high fraction of clays will prevent significant stormwater volume reductions due to their poor infiltration capacity. If stormwater volume reductions are a goal for a detention basin, soils can be amended to improve the capacity for infiltration. Higher infiltration rates will result in larger volumes entering the soils for immediate infiltration, as well as after storm ET losses. The ET rates are also important, as they affect whether soils dry out in time to infiltrate stormwater from the next event.

It is expected that wet ponds and wetland basins or channels might not significantly decrease the volume of runoff because soils suitable for placement of a wet pond or wetland basin will typically exhibit low infiltration capabilities. Otherwise a liner will be necessary to maintain the water quality pool during the wet season. Due to the need to maintain a permanent wet pool for optimal pollutant removal in a constructed wetland, little volume reduction can be expected due to infiltration losses. However, volume reductions would be expected in biofilters due to drier, more permeable soils and complete vegetative cover.

Based on the limited study data available, dry detention basins and biofilters show an average volume reduction of 30% and 38%, respectively, while wet ponds and wetland basins show an average volume reduction of 7% and 5%, respectively (Table 6-1) (Strecker et al., 2004). Based on this analysis, detention basins (dry ponds) and biofilters (vegetated swales, overland flow, etc.) appear to contribute significantly to volume reductions, even though they are not likely designed specifically for this purpose. Assuming a runoff capture efficiency (discussed in the next section) of 80%, a dry detention basin could be expected to reduce stormwater runoff volumes by about 25% on average. The actual volume reduction depends on the infiltration characteristics of the soils and local ET rates.

Table 6-1. Average Volume Losses in Treatment System Components.

BMP Type	Mean Monitored Outflow/Mean Monitored Inflow for Events Where Inflow is Greater Than or Equal to 0.2 Watershed Inches
Detention Basins	0.70
Biofilters	0.62
Media Filters	1.00
Hydrodynamic Devices	1.00
Wetland Basins	0.95
Retention Ponds	0.93
Wetland Channels	1.00

(Source: International Stormwater BMP Database.)

6.8.2 Capture Efficiency

The capture efficiency (percent of stormwater runoff treated) of an on-line volume-based BMP (e.g., detention facility) is primarily a function of the volume of the facility and the hydraulic design of the outlet structure (e.g., brimfull or half-brimfull emptying time of the detention facility). A properly designed storage-based BMP should generally result in capture efficiencies satisfying regulations (e.g., on the order of ≥ 70-90% of the long-term flows from the watershed). Untreated stormwater runoff volumes that bypass the detention facility will therefore generally be less than 30% of the runoff volume, ideally on the order of 10-20%.

For volume-based BMPs, the bypassed, untreated flows occur most often from the tail end of the storms. Depending on the pollutant and the runoff characteristics of the watershed, these bypasses will frequently have lower pollutant concentrations because the majority of accumulated pollutants are discharged earlier in the storm. This higher pollutant loading at the beginning of a storm event is referred to as the “first flush” effect (Sections 3.4 and 3.5). However, for many pollutants and under the varied rainfall conditions found seasonally and geographically around the United States, a first flush effect may not exist. First flush is frequently oversimplified for the complex phenomena of pollutant source loading to runoff.

Design Volume (depth): In simplified sizing approaches frequently used in regulatory environments, the design storage volume of a detention basin is based on an equivalent depth of runoff over the tributary area. The runoff volume (tributary area x runoff depth) for the area is a function of the watershed size and runoff coefficient, and the runoff coefficient is a function of the impervious fraction and soil type(s) in the watershed. Larger design depths (inches over the watershed) result in a larger percentage of the stormwater runoff captured by the basin. However, as design depths become excessively large, only relatively marginal improvements are gained. Given that drain time criteria for the basin remain constant, very large design depths result in high water depths in the basin (e.g., > 4 ft) and are undesirable because the smaller, more frequent storms will pass quickly through the basin without receiving sufficient detention time for sedimentation to occur. A proper design depth provides nearly complete treatment of smaller, more frequent storms, and captures significant portions of larger storms. Proper design of the basin outlet structure is necessary to achieve this goal. Continuous simulation screening results are provided in Appendix D that show percent capture versus design depths for thirty locations in the United States.

Drawdown Rate: The drawdown rate is the outflow rate (cfs), not necessarily constant, at which a detention facility is completely emptied. The drawdown rate is directly related to the design of the outlet structure, which may include weirs, orifices, or combination (see Section 5.6). A simple multistage outlet design may be used to quickly create storage capacity for the next storm event if the next storm occurs before the basin is completely empty (i.e., storm interevent times typically less than 24-48 hours). For instance, one approach may be to empty the top half of the detention basin in one-third of the detention time (from full pool), while the lower half is emptied in the remaining detention time. Slower drawdown of the lower half of the pool promotes effective treatment for smaller storm events that would not completely fill the detention basin. Typical target drawdown times range from 24 to 48 hours to achieve sedimentation and removal of associated pollutants. Short drawdown times (e.g., 24 hours) create storage volume more quickly, resulting in a higher capture efficiency, but do not allow as much time for sedimentation. An appropriate stage-discharge relationship should be based on the average hydraulic residence time of a slug of water passing through the basin at a range of flow conditions needed to remove the target particle size. Longer detention times are appropriate to treat runoff with a large fraction of fine particles. Shorter detention times are more appropriate for stormwater runoff with fewer fines, or in areas with storms that occur in series with short interevent times. A maximum drain time of 36 hours from a brimfull condition is often an appropriate compromise between the removal efficiency of particles and capture efficiency of stormwater runoff volumes. Drawdown at maximum is the seasonal mean time between precipitation events in a watershed. Refer to Chapter 7 for a more detailed discussion of the tradeoffs between stage-discharge and capture efficiency.

Flow Rate: Flow-based treatment systems such as most swales and proprietary devices, are frequently sized based on calculation of a peak flow estimates derived from a design event, unit hydrograph, or rainfall/runoff model. One major disadvantage of flow-based design is that it does not normally account for the volume of the runoff hydrograph. A flow-based system is best sized to capture a required runoff volume (say 80%) by 1) plotting the historical hydrograph, 2) choosing a range of flow rates, and 3) integrating under all the different flow rates. The integration at a particular flow rate would yield the runoff volume (e.g., 80%) to be treated. Based on this runoff hydrograph analysis, the flow-based system could then be sized for the flow rate that would capture the runoff volume to be treated. Flow-based treatment systems are evaluated in Appendix D for thirty locations in the United States.

6.8.3 Pollutant Removal

Currently, the most comprehensive set of BMP water quality performance data is the ASCE/EPA International Stormwater BMP Database (www.bmpdatabase.com). However, even with the significant increase in the amount of data in the database over the last three years, the total number of BMPs in any one category is still relatively small compared to the number of design parameters and other regional factors that could be potentially investigated (Table 6-2). Nonetheless, the consolidation of BMP performance data is beginning to provide a basis for evaluating the actual performance of BMPs and how performance differs between BMP types, design variations, and geographic regions. The following paragraphs discuss some of the conclusions of recent analysis of the BMP database (Strecker et al., 2004).

Table 6-2. Number of Structural BMPs in the International BMP Database.

BMP Category	Number Of BMPs
Structural	
Biofilter (Grass Swales)	32
Detention Basin	24
Hydrodynamic Device	17
Media Filter	30
Percolation Trench/Well	1
Porous Pavement	5
Retention Pond	33
Wetland Basin	15
Wetland Channel	14
Total	**171**
Non-Structural	
Maintenance Practice	28
Total	**28**
Grand Total	**199**

State (Domestic)	Number of BMPs
AL	13
CA	41
CO	4
FL	24
GA	2
IL	5
MD	5
MI	5
MN	7
NC	6
NJ	3
OH	1
OR	3
TX	19
VA	29
WA	20
WI	10
International	
Sweden	1
Canada	1

Options for evaluation of performance data are discussed extensively in Section 8.5 of this project's *Final Research Report*. The most common performance measure used today is "percent removal" of pollutants. However, analyses of data contained in the ASCE/EPA BMP Database have shown that percent removal is a highly problematic method for assessing performance and has resulted in some significant errors in BMP performance reporting (Strecker et. al., 2001). Effluent quality is much less variable than the percent removed (or fraction removed) for BMP studies, as shown in Figure 6-1, which shows box plots by BMP types of the fractions of total suspended solids (TSS) removed and box plots of TSS effluent quality. The box plots present the median, the upper and lower 95 percent confidence intervals of the median, and the 25th and 75th percentiles. For example, the 95% confidence interval for the median wet pond removal is between about 50 and 90 percent (a little better than 0-to-100), while the median effluent quality 95% confidence range is between approximately 11 to 18 mg/L. Furthermore, as has been found previously (Strecker et al., 2001), it appears that percent removal is more or less a function of how "dirty" the inflow is rather than how effective the system is at reducing pollutant concentrations to a target level. Recent analyses of the BMP Database have shown (Strecker et al., 2004) that for some pollutants percent removals are not significantly different between BMP types while effluent concentrations are significantly different. For example, Figure 6-1 shows no significant difference between the TSS percent removals for bioswales and detention basins, but bioswales have a significantly lower effluent quality.

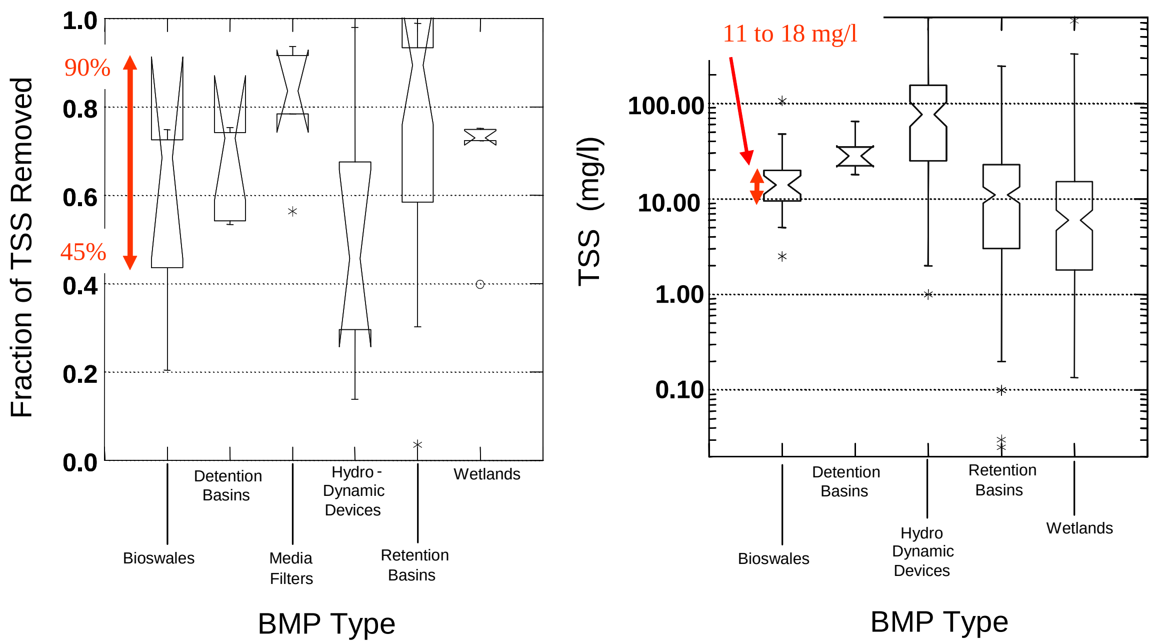

Figure 6-1. Box plots of the fractions of total suspended solids (TSS) removed and of effluent quality of selected BMP types, by BMP Study.

Median effluent quality for various BMPs is shown in Table 6-3 for common target constituents. The data are from the International Stormwater BMP Database. Data summaries by typical stormwater pollutants are also provided in the Pollutant Fact Sheets (Appendix A). The degree of pollutant removal depends on the pollutant species/form, and the extent to which appropriate UOPs occur within the treatment system. In addition, design features such as pond area or use of a forebay can significantly affect effluent quality.

Table 6-3. Median of Average Effluent Concentrations of Treatment System Components.

Constituents	Point of Discharge	Detention Pond	Wet Pond	Wetland Basin	Biofilter	Media Filter	Hydrodynamic Devices
Suspended Solids (mg/L)	Influent	87.73 (48.4-159.1)	88.38 (48.9-159.7)	82.12 (65.7-102.7)	51.95 (22-123)	61.14 (45.4-82.4)	110.74 (51.1-240.1)
	Effluent	41.35 (30.8-55.5)	19 (12.9-28.0)	19.68 (16.6-23.4)	24.6 (15.0-40.3)	25.47 (14.7-44.3)	40.34 (18.4-88.7)
Total Cadmium (µg/L)	Influent	2.3 (1.9-2.9)	0.55 (0.1-2.6)	xx	0.58 (0.3-1)	0.4 (0.2-0.8)	1.69 (1.4-2.1)
	Effluent	1.3 (0.8-2.2)	0.31 (0.05-2.0)	xx	0.25 (0.21-0.34)	0.31 (0.16-0.59)	1.65 (1.05-2.6)
Dissolved Cadmium (µg/L)	Influent	xx	xx	xx	0.26 (0.1-0.9)	0.29 (0.2-0.4)	0.72 (0.4-1.3)
	Effluent	0.41 (0.22-0.76)	xx	xx	0.22 (0.11-0.43)	0.24 (0.18-0.33)	0.93 (0.27-3.2)
Total Copper (µg/L)	Influent	32.28 (22.7-46)	17.89 (7.4-43)	xx	21.8 (11.6-40.9)	15.29 (12.4-18.8)	21.43 (14.7-31.2)
	Effluent	18.9 (16.6-21.5)	6.92 (4.7-10.3)	xx	10.01 (5.6-17.9)	9.81 (8.1-11.8)	14.13 (11.1-18.1)
Dissolved Copper (µg/L)	Influent	12.11 (8-18.3)	8.87 (5.4-14.6)	xx	12.32 (6.5-23.4)	8.83 (6.7-11.6)	12.07 (3.7-39.5)
	Effluent	14.72 (10.4-20.9)	5.09 (3.1-8.3)	xx	7.66 (4.7-12.5)	7.95 (6.6-9.7)	8.63 (3.3-22.9)
Total Chromium (µg/L)	Influent	9.45 (9.5-9.5)	7.31 (2.7-19.6)	xx	2.46 (1.1-5.5)	2.78 (1.8-4.3)	xx
	Effluent	2.85 (1.7-4.8)	1.78 (0.5-6.7)	xx	2.18 (1.2-4.0)	1.46 (0.9-2.3)	xx
Total Lead (µg/L)	Influent	69.2 (33.6-142.5)	33.34 (10.2-109.3)	12.63 (3.8-42)	19.56 (7.4-51.6)	15.61 (9.3-26.1)	21.96 (10.6-45.7)
	Effluent	15.02 (9.5-23.8)	6.68 (2.9-15.6)	3.25 (1.9-5.6)	6.95 (4.2-11.7)	5.5 (3.5-8.6)	12.98 (4.2-40.2)
Dissolved Lead (µg/L)	Influent	3.4 (2-5.8)	9.48 (0.9-101.4)	xx	2.5 (0.9-6.9)	2.18 (1.6-3.1)	1.87 (1.1-3.1)
	Effluent	2.33 (1.7-3.3)	4.16 (2.0-8.9)	xx	1.35 (0.5-3.6)	1.42 (1.0-1.9)	2 (0.6-6.5)
Total Zinc (µg/L)	Influent	273.84 (177.7-421.9)	75.29 (44-128.9)	164.27 (54.6-494.1)	129.17 (57.3-291.3)	121.8 (72.6-204.3)	167.1 (123.1-226.8)
	Effluent	85.26 (50.6-143.7)	28.63 (21.4-38.3)	118.73 (32.8-429.5)	39.44 (28.2-55.2)	64.96 (45.3-93.2)	89.66 (74.4-108.1)
Dissolved Zinc (µg/L)	Influent	48.79 (22.7-104.8)	57.36 (20.1-163.4)	xx	67.39 (33.8-134.4)	71.66 (41.3-124.4)	48.56 (28-84.3)
	Effluent	43.99 (20.0-96.6)	16.89 (2.6-109)	xx	31.96 (26.7-38.3)	57.14 (37.7-86.6)	45.17 (29.6-68.9)
Total Phosphorus (mg/L)	Influent	0.4 (0.3-0.5)	0.53 (0.3-0.9)	2.91 (1.9-4.6)	0.19 (0.1-0.4)	0.25 (0.2-0.4)	0.79 (0.3-2.2)
	Effluent	0.3 (0.2-0.44)	0.16 (0.12-0.21)	0.15 (0.07-0.33)	0.32 (0.24-0.43)	0.14 (0.11-0.17)	0.19 (0.07-0.51)
Dissolved Phosphorus (mg/L)	Influent	xx	0.2 (0.1-0.4)	0.06 (0-0.1)	xx	xx	xx
	Effluent	xx	0.07 (0.04-0.13)	0.07 (0.03-0.18)	xx	xx	xx
Total Nitrogen (mg/L)	Influent	xx	1.49 (0.6-3.6)	2.56 (1.6-4)	0.58 (0.3-1)	xx	xx
	Effluent	xx	1.17 (0.77-1.78)	2.42 (1.46-4.0)	0.69 (0.37-1.29)	xx	xx
Nitrate-Nitrogen (mg/L)	Influent	0.89 (0.5-1.6)	1.15 (0.3-5.1)	0.63 (0.4-1.1)	0.31 (0.2-0.6)	0.55 (0.4-0.8)	xx
	Effluent	0.64 (0.37-1.09)	0.48 (0.11-2.05)	0.46 (0.16-1.28)	0.5 (0.36-0.68)	0.82 (0.68-0.97)	xx
TKN (mg/L)	Influent	1.99 (1.6-2.5)	1.06 (0.8-1.4)	1.23 (1-1.6)	2.27 (1.8-2.9)	2.2 (1.7-2.9)	6.37 (2.3-17.3)
	Effluent	1.87 (1.46-2.39)	0.84 (0.68-1.04)	1.33 (0.84-2.11)	1.6 (1.42-1.8)	1.79 (1.45-2.2)	4.68 (1.97-11.12)

(Source: International Stormwater BMP Database October , 2004; all units in mg/L; values in parenthesis are the 95% confidence intervals about the median).

xx - Lack of sufficient data to report median and range.

6.8.3.1 Suspended Solids

Larger suspended solids can be removed effectively by gravitational sedimentation, screening or surficial straining. For most well designed BMPs that incorporate these UOPs, the median effluent concentrations range from 20 to 25 mg/L, provided the concentration and characteristics (e.g., particle size distributions) of influent suspended solids do not significantly deviate from “typical” stormwater. Well designed treatment systems that incorporate wet pools and wetland vegetation typically exhibit good effluent quality for suspended solids. Based on currently available data, these BMPs can typically achieve effluent concentrations of around 20 mg/L. Well designed biofilters and media filters also perform well in achieving low effluent suspended solids concentrations.

The presence of a permanent wet pool is a feature of a wet pond/wetland system. Incorporating even a small permanent wet pool can significantly improve the sediment removal performance of a BMP by providing long periods of retention during smaller storms. Long retention times during small events allow for appreciable sediment removal compared to dry facilities that typically have very limited detention times during small events. Generally, settleable solids comprised of inorganic particles in the 25-75 µm range are effectively removed by quiescent gravitational sedimentation.

For biofilters and media filters, gravity settling and filtration are the primary removal mechanisms for suspended sediments. Direct filtration can usually be effectively accomplished at concentrations less than 50 mg/L, and generally requires some level of pretreatment in urban runoff, where solids concentrations are frequently above 100 mg/L and can exceed 1,000 mg/L depending on the site, loading, and hydrology. Generally, suspended inorganic particles less than 25 µm require some natural or enhanced coagulation/flocculation followed by sedimentation and/or filtration.

Based on the available data, the central tendency of TSS effluent concentrations is significantly higher (i.e., poorer effluent quality) for dry detention basins (which drain after each event and generally lack a significant littoral zone) and hydrodynamic BMPs (flow-through systems that rely on centrifugal forces to provide treatment). However as noted, dry detention basins have been shown to provide considerable reduction in effluent volume (up to 30%), which may translate to lower total mass loading of TSS downstream.

6.8.3.2 Trace Metals

The important forms of trace metals from a treatability and regulatory perspective are total, dissolved, and particulate-bound metals. If bound to organic or inorganic particulates, viable unit operations include sedimentation and filtration either as separate unit operations or in combination with coagulation/flocculation as pretreatment to these operations. If present as a dissolved complex, precipitation could be effective. If present as a dissolved ionic species such as Cu^{2+}, Pb^{2+}, or Zn^{2+}, surface complexation (including adsorption) could be effective. Based on effluent quality, well designed wet ponds, biofilters, and media filters can provide better effluent quality compared to detention pond and hydrodynamic devices (Table 6-3). BMPs that are effective in removing trace metals also are typically good at removing fine particulates.

6.8.3.3 Nutrients

Treatability for phosphorus is a function of whether phosphorus is present in particulate or dissolved form. In dissolved form, phosphorus may readily undergo surface complexation reactions, sorption, or precipitation (see Pollutant Fact Sheets in Appendix A). Uptake by vegetation and microbes is another mode by which dissolved phosphorus is effectively removed. Media or soils containing iron, aluminum or hydrated Portland cement can be very effective at removing dissolved phosphorus species through surface complexation or precipitation. If bound to organic or inorganic particles, viable UOPs include sedimentation and filtration either alone, or in combination with pretreatment using coagulation/flocculation.

As shown in Table 6-3, media filters, wet ponds, and wetland basins report the lowest median effluent total phosphorus concentrations, although only wet ponds have a statistically significant difference between median influent and effluent values (i.e., the BMP affected total phosphorus concentrations). While median effluent levels for dissolved phosphorus are the lowest for wetland basins, the available data are insufficient to reliably differentiate the performance of various BMPs.

Nitrogen compounds exist in dissolved form and as particulate-bound species. Treatability success for nitrogen species, as with other constituents in stormwater, is highly dependent on the form and species of nitrogen present. Treatability for nitrogen also depends on the presence of specific bacteria that mediate nitrogen transformations. Physical operations such as sedimentation have played an insignificant role with respect to treatment of nitrogen as compared to microbially mediated transformations. Microbial decomposition of organic matter mineralizes nitrogen to ammonia, which can be oxidized to nitrite and nitrate. Nitrate can be reduced to nitrogen gas by anaerobic bacteria, for complete removal from the system. Median effluent quality of total nitrogen, total Kjeldahl nitrogen (TKN), and nitrate-nitrogen is summarized in Table 6-3. However, available data on removal of nitrogen species are insufficient to draw definitive conclusions about BMP performance based on averaged effluent concentrations.

Filters, ditches, and dry ponds typically exhibit poor nitrate removal, and in many cases have been shown to export nitrate. In these BMPs, organic nitrogen is converted to nitrate in the mineralization and nitrification processes; however the aerobic conditions are not favorable for denitrification. Thus, theses BMPs may export more nitrate than is present in the influent. Conversely, in wet ponds and wetland basins, plants, algae and other microorganisms take up nitrate as an essential nutrient. However, nitrogen is also released back into the system upon death or decay of the organisms.

6.9 Relative Cost Assessment

Estimating the costs of stormwater treatment systems, including annual operation and maintenance costs, is one of the more challenging aspects of stormwater planning and management. The following subsections present some simplified methodologies for estimating these costs.

6.9.1 Life Cycle Costs

Life cycle costs should be evaluated for candidate BMPs. The present value of the life cycle cost for a treatment system is equal to the sum of the following costs: 1) initial construction, 2) land, 3) the present worth of the total annual operating and maintenance costs, and 4) the present worth of the salvage value of the system. Annual life cycle costs can be calculated as the sum of the following costs: 1) the amortized value of initial construction, 2) amortized land value, 3) average annual operating and maintenance costs, and 4) the amortized salvage value.

Equations 6-1, 6-2, 6-3 can be found in any text on engineering economics, and can be used to calculate the present worth of annual operation and maintenance (O&M) costs, present worth of the salvage value, and an equivalent annual value of the present worth, respectively. The service life (n time units) of the system and an associated interest rate (*i*, per time unit) must be estimated to calculate costs.

Present worth (P) of annual O&M costs (A)

$$P = A\left(\frac{(1+i)^n - 1}{i(1+i)^n}\right) \qquad [6\text{-}1]$$

Present worth (P) of the salvage value (F)

$$P = \frac{F}{(1+i)^n} \qquad [6\text{-}2]$$

Equivalent annual value (A) of a present worth (P).

$$A = P\left(\frac{i(1+i)^n}{(1+i)^n - 1}\right) \qquad [6\text{-}3]$$

Note that *i* is the interest rate per compounding time period (A) over the service life of the system (n). For instance, if interest rates were compounded monthly, then *i* is the annual interest rate divided by 12, and n is equal to 12 × the number of years. The costs provided in this document are converted to July 2004 dollars using the consumer price index (CPI) shown in Table 6-4. The CPI is used because it is in the public domain and provides a good composite index across construction, operation and maintenance, and land value changes. The Engineering News Record construction cost index is another popular index. It is available for a small charge from the Engineering News Record (http://enr.construction.com/features/conEco/subs/default-city.asp?referid=1850). Equation 6-4 illustrates how the CPI is used to estimate costs for a particular year, X, based on costs for a different year, Y.

Costs(Year-X) = Costs(Year-Y) x CPI(Year-X) / CPI(Year-Y) [6-4]

Table 6-4. Consumer Price Index (CPI) for 1994 to 2004.

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL
2004	185.2	186.2	187.4	188	189.1	189.7	189.4
2003	181.7	183.1	184.2	183.8	183.5	183.7	183.9
2002	177.1	177.8	178.8	179.8	179.8	179.9	180.1
2001	175.1	175.8	176.2	176.9	177.7	178	177.5
2000	168.8	169.8	171.2	171.3	171.5	172.4	172.8
1999	164.3	164.5	165	166.2	166.2	166.2	166.7
1998	161.6	161.9	162.2	162.5	162.8	163	163.2
1997	159.1	159.6	160	160.2	160.1	160.3	160.5
1996	154.4	154.9	155.7	156.3	156.6	156.7	157
1995	150.3	150.9	151.4	151.9	152.2	152.5	152.5
1994	146.2	146.7	147.2	147.4	147.5	148	148.4
	AUG	**SEP**	**OCT**	**NOV**	**DEC**	**AVG**	
2004	189.5	189.9	190.9	191	190.3	188.9	
2003	184.6	185.2	185	184.5	184.3	183.96	
2002	180.7	181	181.3	181.3	180.9	179.88	
2001	177.5	178.3	177.7	177.4	176.7	177.1	
2000	172.8	173.7	174	174.1	174	172.2	
1999	167.1	167.9	168.2	168.3	168.3	166.6	
1998	163.4	163.6	164	164	163.9	163	
1997	160.8	161.2	161.6	161.5	161.3	160.5	
1996	157.3	157.8	158.3	158.6	158.6	156.9	
1995	152.9	153.2	153.7	153.6	153.5	152.4	
1994	149	149.4	149.5	149.7	149.7	148.2	

Source: http://inflationdata.com/inflation/Consumer_Price_Index/HistoricalCPI.aspx
Note: historical CPI data back to 1913 are available at this web site.

6.9.2 Previous Cost Studies

The results of numerous studies of BMP costs during the past 30 years have been summarized in recent reports. Heaney et al. (1999) developed wet-weather control costs as part of a larger study on BMP cost-effectiveness for the USEPA. Results of this study are summarized in Sample et al. (2003). These documents include summaries of the cost information presented by Young et al. (1996) who summarized earlier cost data with emphasis on highway costs. Numerous earlier cost studies were summarized by USEPA (1999f). A report for Caltrans (2001) compares the costs of 39 BMPs built as test projects for them with other studies, including significant efforts by the Southeastern Wisconsin Regional Planning Council (1991) and Brown and Schueler (1997). The Caltrans BMP costs were much higher than other reported costs. The primary reason seems to be that these BMPs were single purpose water quality retrofits in highly developed expressways. Thus, the Caltrans data are considered to be atypical and are not included in this analysis. The cost data presented herein are taken from the following sources:

1. City of Austin, TX (Caltrans, 2001)
2. City of Portland, OR (Caltrans, 2001)
3. Delaware DOT (Caltrans, 2001)
4. Florida Department of Environmental Protection (Caltrans, 2001)
5. King County, WA (Caltrans, 2001)
6. Maryland State Highway Administration (Caltrans, 2001)

7. Oregon DOT (Caltrans, 2001)
8. Prince George's County, MD (Caltrans, 2001)
9. Snohomish County, WA (Caltrans, 2001)
10. Virginia DOT (Caltrans, 2001)
11. Washington State DOT (Caltrans, 2001)
12. Wossink and Hunt (2003)
13. Minton (2005)

These cost data were compiled into a single spreadsheet database for the analysis that follows.

6.9.3 Construction Costs

Construction costs are normally estimated using one of two methods:

1. Cross-sectional analysis of comparable projects.
2. Process analysis of designs of various sizes by a design team. RSMeans® data (http://www.rsmeans.com) are based on this approach with some calibration against recently constructed projects.

The process analysis approach should be more reliable since it is performed under more carefully controlled conditions wherein all parameters, but one, are held constant (e.g., the cost of a house as a function of square footage). The process approach is highly recommended and should be used once candidate BMPs have been identified. The results of both approaches will be presented in this section.

6.9.3.1 Cross-Sectional Analysis of Comparable Projects

The traditional way to summarize cost estimating data is to approximate the total cost using a single variable power function as shown in Equation 6-5. This power function is linear in the log-transform. The two parameters can be estimated from a log-log graph or found using linear regression on the log-transformed data, or using nonlinear regression on the untransformed data. Finding the two parameters for the power function is easily accomplished using built in trend line functions available in standard spreadsheet software.

$$C = \alpha_0 x^{\alpha_1} \qquad [6\text{-}5]$$

Where: $C =$ total cost, $,
$x =$ independent variable that is a measure of size,
$\alpha_0 =$ coefficient, and
$\alpha_1 =$ exponent.

The exponent, α_1, represents the economies of scale, which in general terms, are factors that cause the average cost of production to decrease with increased output. If $\alpha_1 < 1.0$, then unit costs decrease as size increases. A generic economies of scale factor that has been used for years is $\alpha_1 = 0.6$ (Peters and Timmerhaus, 1980). When $\alpha_1 = 1$, the power function simplifies to a linear relationship and no economies of scale are present. If $\alpha_1 > 1$, then diseconomies of scale exist.

The power function approximation offers a way to replace a cost database with a single equation. It also satisfies the condition that total cost is zero if size is zero. However, this simple approximation may be inaccurate. Also, total cost is seldom a function of only one explanatory variable. For multiple variables, the estimated cost can be expressed in a general form as:

$$C = f(x_1, x_2, \ldots x_i, \ldots x_n) \qquad [6\text{-}6]$$

Where: $C =$ total cost, and
$x_i =$ i^th independent variable

If a database of total costs as a function of *n* explanatory variables is available, an approximating equation can be developed using several multiple regression approaches. The drawback to this approach is that the relationship of total cost to explanatory variables is seldom this simple. For stormwater treatment systems, virtually all cost data are presented as a function of one of the following measures of size:

1. Contributing drainage area
2. Volume of the TSC
3. Design flow rate into the TSC

A cost analysis by Heaney and Lee (2005) using data from the above sources is summarized in Table 6-5. This database consists of 106 BMPs. Most data are for systems that incorporate storage BMPs (70 sites), with wetlands comprising the single largest category (25 sites). The median drainage area for the sites is about 5 acres, but the drainage areas are highly variable. Facilities with storage as the primary TSC tend to drain much larger areas. Obviously, drainage area imperviousness has a major impact on the hydrology of a site, and therefore system sizing and performance. However, for this cost analysis, only information on total drainage area was available, not percent imperviousness. The runoff volume expressed in watershed inches reflects the variation in sizing for the facilities examined and in the information presented here; it ranges from 0.22 to 1.24 inches with a median of 0.78 inches. The median is used as the measure of central tendency due to wide variability in the data. The estimated unit costs per acre treated vary from about $3,700 for extended detention ponds to over $90,000 for Delaware sand filters. A better measure of drainage area is DCIA because it is the primary cause of runoff from the more frequent storms (Lee and Heaney, 2003). Wossink and Hunt's (2003) results show the same wide variability. If the price per gallon ($/gal) of water stored is used as the metric of performance, then the rankings are similar. Unit costs, in $/gal, range from $0.08 for wetlands to nearly $3.00 for sand filters. An important reason why wetlands have the lowest unit cost is that they are the largest controls and reflect the expected economies of scale for larger control units. Analysis of the effect of size on total costs showed wide scatter in the data with little evidence of economies of scale. However, a more accurate way to explore economies of scale is to use process-based data as will be described in the next section.

Table 6-5. Construction Costs as a Function of Service Area and Design Volume.

Control	Samples	Median Drainage Area, ac.	Median Runoff in./acre	Caltrans Median $/acre treated	NC State Median $/acre treated	Caltrans Median $/gallon	Caltrans Median 1,000 gal.
Compost Filter	11	2.70	0.22	$ 10,000		$ 0.60	68.4
Detention Pond, Extended	23	16.50	0.94	$ 3,739		$ 0.12	273.0
Detention Pond, Wet	22	36.70	0.43	$ 7,366	$ 4,600	$ 0.32	717.0
Detention Pond, Wetland	25	47.40		$ 7,444	$ 666	$ 0.08	1787.7
Infiltration Trench	8	4.36	0.78	$ 12,476		$ 0.62	14.1
Sand Filter, Austin	9	6.01	0.62	$ 14,952	$ 39,600	$ 2.86	68.8
Sand Filter, Delaware	4	1.30	1.24	$ 90,725	$ 39,600	$ 2.87	45.8
Swale	4	3.53	0.86	$ 11,238		$ 0.36	50.7
Total	106						
Median	10	5.18	0.78	$ 10,619	$ 22,100	$ 0.48	69

*All costs are in July 2004 dollars.

6.9.3.2 Process Analysis of Control Costs

Process analysis of costs is done by having a single group of professionals design a control with all parameters fixed except one measure of size. This approach should provide more accurate information because all of the assumptions are controlled. The limitation of this approach is that it is more expensive and has not been done for wet-weather water quality controls. A good indicator of stormwater treatment costs can be obtained by evaluating similar controls such as storage devices and wastewater treatment plants using published estimates from companies such as RS Means.

Construction costs for four categories of water storage tanks are shown in Table 6-6. These data can be closely approximated by power functions of the form:

$$\text{Construction cost (\$1,000)} = \text{Cost/Volume} = a \times \text{Volume (1,000 gal.)}^{b} \qquad [6\text{-}7]$$

The average cost per gallon is then:

$$\text{Average cost (\$/gallon)} = a \times (\text{Volume (1,000 gal.)}^{b-1} \qquad [6\text{-}8]$$

For example, the average cost of a 100,000 gallon elevated storage tank is (Table 6-6):

Average cost = $19.394*100^{0.5329-1}$ = \$2.26/gallon.

Strong economies of scale exist for storage tanks with a range of exponents from 0.48 to 0.68. As expected, elevated tanks cost more per gallon than ground level tanks.

Table 6-6. Construction Costs for Water Storage Tanks Based on 2003 RS Means Data.

			Parameters*			Capacity in 1,000 gal.		
	Minimum	Maximum				10	100	1000
Storage	1,000 Gal.	1,000 Gal.	a	b	R^2	$/gal.	$/gal.	$/gal.
Elevated	50	1000	19.394	0.5329	0.982		$2.26	$ 0.77
Ground, concrete	100	10000	8.189	0.6185	0.99	$ 3.40	$1.41	$ 0.59
Ground, steel	250	10000	3.38	0.6804	0.982	$ 1.62		$ 0.37
Underground, small	0.55	15	2.3	0.482	0.975	$ 0.70		

*Parameters in the equation Total Cost ($1,000) = a*Capacity in 1,000 gallons^b.

It is instructive to compare unit costs in Table 6-6 with the treatment system costs in Table 6-5. Sand filters are in the 50,000 gallon size range and have a unit cost of about \$2.86/gallon. This estimate is similar to the unit cost of a ground level concrete storage tank. The economies of scale factor is also estimated for packaged wastewater treatment plants. The economies of scale factor is about 0.66.

6.9.3.3 Conclusions on Treatment System Construction Costs

A relatively high variability exists in the reported construction costs of stormwater treatment facilities. Key sources of this variability include:

1. Most of these designs are multipurpose and also provide flood control and drainage. Thus, only a portion of the cost is assignable to water quality control costs.
2. Large regional variability in precipitation and runoff patterns.
3. Large variability in the cross-sectional data on how the costs are calculated.

The following conclusions can be used for preliminary cost estimating purposes.

1. The economies of scale factor for these controls is in the range of 0.65-0.70.
2. The price per gallon is a more reliable measure of average costs than \$/acre since the latter measure does not correct for how much of the drainage area is directly connected.
3. First approximations of unit costs can be made using the following estimates:
4. Volume based BMPs: \$0.10-\$0.30/gallon.
5. Infiltration BMPs and swales: \$0.30-\$0.60/gallon.
6. Filtration BMPs: \$2.50-\$3.00/gallon.

6.9.4 Land Costs

Land costs can have a major impact on stormwater treatment facility costs. They may be zero for subsurface controls, or where the land is free. Land is "free" when it is in a right of way, easement, or it is some type of public land such as a park. However, the use of public parcels for treating highway runoff will have environmental and political constraints that may significantly affect the actual cost of obtaining the parcel for stormwater treatment. At the other end of the spectrum, land cost can be the market value based on alternative uses of the land if it is not used as a stormwater control. In between these two extremes are a myriad of assumptions that can be made as to how this land should be valued. Heaney et al. (1999) and Sample et al. (2003) discuss these questions. No simple answer exists. The question of land valuation is becoming even more relevant as interest in LID increases. A key component of LID is to locate BMPs onsite by integrating them into the existing landscape. Thus, one could argue that a significant portion of landscaping costs should be included in stormwater treatment system costs or that onsite treatment is very inexpensive since the landscaping is already provided.

The following first approximations for land costs can be used in the analysis of urban areas:

- Unimproved land: \$25,000-\$50,000 per acre.
- Improved land with infrastructure for residential development: \$75,000-\$200,000 per acre.
- Improved land for commercial development: \$100,000-\$300,000 per acre.
- High density urban land: \$500,000-\$2,000,000 per acre.

These costs represent the purchase price of land. However, land can have a significant resale value after the life of the project. Thus, one could argue that the return on selling the land will offset the initial cost, and so land has no net cost. A perhaps more reasonable assumption is that the net cost for land is the return forgone while it is being used as part of a treatment facility. The annual operation cost is equal to the land purchase price times the annual interest rate.

Land costs affect how much land is used for the various treatment facilities. Claytor and Schueler (1996), Wossink and Hunt (2003), and Pack (2004) provide current information on land use of controls relative to the drainage area treated as shown in Table 6-7. These ratios are derived from data on in-place BMPs and simulations of the expected design ranges for filter strips.

Table 6-7. Ratio of Control Area to Drainage Area for BMPs.

	Control area/drainage area		
BMP	Min. %	Max. %	Source
Wet pond	1	5	Wossink & Hunt 2003
Infiltration basin	2	3	Claytor & Schueler 1996
Stormwater wetland	1.5	6.5	Wossink & Hunt 2004
Sand filter	0	3	Claytor & Schueler 1996
Bioretention	2	7	Wossink & Hunt 2005
Grass Swale	10	20	Claytor & Schueler 1996
Filter Strips	25	100	Pack 2004

The design depth of the treatment system also affects land costs. Table 6-8 shows an example of the expected impact of land costs on a wet detention system with an average depth of 4 feet versus a wetland with an average depth of 1.5 feet. Land costs are assumed to be $100,000/acre in both cases. The wetland has a lower construction cost per gallon than a detention basin. However, it is only able to store water to a depth of 1.5 feet instead of 4.0 feet for the detention basin. Thus, it requires more land to store the same volume of water. In this case, the land cost doubles the unit cost of the wetland making it only slightly less expensive than the detention basin.

Table 6-8. Effect of Land Costs on Total BMP Costs.

Item	Detention Basin	Wetland
Volume, gallons	100,000	100,000
Volume, ft^3	13,369	13,369
Assumed depth, ft.	4	1.5
Area, acres, =	0.077	0.205
Extra area, %	20%	20%
Area, acres, =	0.092	0.246
Construction $/gallon	$ 0.50	$ 0.25
Total construction cost, $	50,000	25,000
Land cost, $/acre	$ 100,000	$ 100,000
Land cost, $	$ 9,207	$ 24,553
Added cost for land, %	18.4%	98.2%
Total cost,construction + land, $	59,207	49,553
Total $/gallon	$ 0.59	$ 0.50

6.9.5 Operation and Maintenance Costs

Wossink and Hunt (2003) summarize basic operation and maintenance activities based on data collected in the Mid-Atlantic States. The results are listed below.

- Wet Pond: Mowing banks (monthly, seasonal). Outlet/inlet inspection (after large events). Removing vegetation from outlet (varies). Forebay dredging (0-3 times over life of pond).
- Stormwater Wetland: Harvest and replanting of wetland vegetation (0-1 times over life of wetland). Outlet/inlet inspection (after large events). Removing vegetation from outlet (varies). Forebay dredging (0-3 times over life of pond).
- Bioretention Area: Pruning shrubs and trees (0-2 times per year). Mowing (monthly, seasonal). Weeding (monthly, seasonal). Re-mulching (1-2 times per year). Replanting shrubs (0-1 times over life of bioretention area). Removing sediment accumulation (1-2 times over initial life of practice). Underdrain inspection (1 time per year).
- Sand Filter: Dredging sedimentation chamber (1 time annually to 1 time every three years). Removing built up debris from sand chamber (2-3 times per year initially, 1 time per year thereafter). Outlet inspection (1 time per year). Underdrain inspection (1 time per year).

O&M costs are often expressed as a percentage of construction costs. USEPA (1999f) summarizes available estimates of annual O&M costs as a function of initial construction costs as shown in Table 6-9. Review of the more recent literature does not indicate that better estimates are available.

Table 6-9. BMP O&M Costs as a Percentage of Construction Costs.

Source: USEPA, 1999f.

Type	% of Construction $
Detention basins	<1
Retention basins	3 to 6
Constructed wetland	3 to 6
Infiltration trench	5 to 20
Infiltration basin	1 to 10
Sand filter	11 to 13
Bioretention	5 to 7
Grass swale	5 to 7
Filter strip	320/acre maintained

6.9.6 Salvage Values at End of Project

Salvage value can be negative (e.g., the system has to be removed) when the accumulated sediment is deemed to be hazardous and must be removed at great cost). It can also be positive, e.g., the land value appreciates significantly during the project. It is very difficult to estimate these future events. Thus, salvage values are assumed to be zero for evaluating life cycle costs.

6.9.7 Total Life Cycle Cost

The equivalent uniform annual cost of the construction cost plus land cost plus O&M costs will be determined in this section based on the data presented above. All systems are assumed to last for 20 years and an interest rate of 5% per year is used. The results for the indicated scenario are shown in Table 6-10.

The data from the previous sections on construction, O&M, and land costs were used to estimate the life cycle costs (LCC) for a scenario of a control with a storage capacity of 100,000 gallons. Construction costs are based on the median $/gallon for each category. The present value of annual O&M costs can be determined by multiplying the annual O&M as a percentage of construction costs by the Present Worth Factor for a Uniform Series at an interest rate of 5% and a 20-year service life (Equation 6-1). This factor is 12.46. Thus, for compost filters with an annual percentage of 10, the present worth of these annual O&M costs is 1.246 times the construction costs. The storage depth for each BMP is used to estimate the land area. The cost of the land is assumed to be the opportunity foregone during this 20-year period or $5,000 per year. The present worth of 20-years of equal payments of $5,000 per year is $62,311. Then, the total costs are the sum of construction costs, O&M costs and land costs. Lastly, the full life cycle cost (LCC) per gallon is shown.

Table 6-10. Life Cycle Cost for Eight BMPs with a Storage Capacity of 100,000 Gallons.

N, years =	20
Interest, i/yr. =	0.05
Present worth factor uniform series =	12.4622
Assumed size, gallons =	100,000
Net land cost, $/acre =	$ 62,311

Control	Const. Cost $/gallon	Total Const. Cost, $	O&M % of Const. $/year	PW of O&M as % of Const. $	Storage Depth, ft.	Extra Land, %	Land Acres	Net Land Cost, $
Compost Filter	$ 0.60	$ 60,000	10%	124.6%	1	0.1	0.338	$ 21,061
Detention, Extended	$ 0.12	$ 12,000	3%	37.4%	4	0.2	0.092	$ 5,733
Detention, Wet	$ 0.32	$ 32,000	3%	37.4%	4	0.2	0.092	$ 5,733
Detention, Wetland	$ 0.08	$ 8,000	5%	62.3%	1.5	0.2	0.246	$ 15,329
Infiltration Trench	$ 0.62	$ 62,000	6%	74.8%	1	0.1	0.338	$ 21,061
Sand Filter, Austin	$ 2.86	$ 286,000	12%	149.5%	8	0.1	0.042	$ 2,617
Sand Filter, Delaware	$ 2.87	$ 287,000	12%	149.5%	1	0.1	0.338	$ 21,061
Swale	$ 0.36	$ 36,000	6%	74.8%	1	0.1	0.338	$ 21,061

	Total Present Worth of LCC				LCC
Control	Const.$	Land $	O&M $	Total $	Total $/gallon
Compost Filter	$ 60,000	$ 21,061	$ 74,773	$ 155,834	$ 1.56
Detention, Extended	$ 12,000	$ 5,733	$ 4,486	$ 22,219	$ 0.22
Detention, Wet	$ 32,000	$ 5,733	$ 11,964	$ 49,696	$ 0.50
Detention, Wetland	$ 8,000	$ 15,329	$ 4,985	$ 28,313	$ 0.28
Infiltration Trench	$ 62,000	$ 21,061	$ 46,359	$ 129,421	$ 1.29
Sand Filter, Austin	$ 286,000	$ 2,617	$ 427,703	$ 716,320	$ 7.16
Sand Filter, Delaware	$ 287,000	$ 21,061	$ 429,198	$ 737,259	$ 7.37
Swale	$ 36,000	$ 21,061	$ 26,918	$ 83,979	$ 0.84

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CHAPTER 7 SIZING OF TREATMENT SYSTEM AND LID ELEMENTS

7.1 Introduction

Selection and sizing of wet-weather controls (BMP and LID facilities, hereinafter referred to as BMPs) are strongly influenced by the nature of the inflow to such facilities whether for application to stormwater runoff from highways, broader application to urban runoff in general, and for many combined sewer overflows. Probable storm event volumes must be considered as well as peak flows, event duration, hydrograph recession, interevent duration, and influences of a possible first-flush – or more generally, the temporal distribution of pollutant concentrations and loads. Strecker et al. (2001) advocate evaluation of BMP effectiveness by addressing four questions:

1. How much stormwater runoff is prevented (e.g.., infiltrated, evaporated) on-site (hydrological source control)?
2. How much of the runoff that occurs is treated by the BMP?
3. Of the runoff treated, what is the effluent quality?
4. What are downstream impacts?

This chapter addresses the first three questions through continuous simulation of quantity and quality processes for standardized urban catchments across the United States and is based on work performed to aid in preliminary *screening* for BMP sizing for highway and urban drainage engineers. These results are intended to provide *screening methods* that are more geographically focused than those presented by Urbonas and Stahre (1993) and WEF-ASCE (1998), for instance. Two continuous models were used for this purpose:

- The EPA Storm Water Management Model, SWMM, both version 4.4h (http://ccee.oregonstate.edu/swmm/) and version 5 (http://www.epa.gov/ednnrmrl/swmm/).
- A spreadsheet model developed by Heaney and Lee (2005) that performs many of the same functions as SWMM and also includes optimization features.

A general discussion of methods and models appropriate for hydrologic evaluation of highway runoff is provided in Chapter 10 of the *Final Research Report* for this project. While SWMM and spreadsheet methods are highlighted here, other models may be suitable for this type of evaluation. Table 7-1 provides a brief comparison of the chosen models to two other popular hydrologic models, HSPF and STORM.

Table 7-1. Comparison of Stormwater Models.

Model	Flow Routing	Pollutant Routing	Time Scale	Optimization
HSPF	Hydrologic	Complete-mix	Event & continuous	Indirect iteration
STORM	Simple storage	Complete-mix	Event & continuous	Indirect iteration
SWMM	Hydrologic & hydraulic	Complete-mix & plug-flow	Event & continuous	Indirect iteration
CU/UF Spreadsheet Model	DS and infiltration	Plug-flow	Event & continuous	Direct integration with optimization tools

Several options are available for hydrologic regionalization based on statistical analysis of rainfall data. However, BMP selection is fundamentally based on stormwater and pollutant inflow to the BMP rather than on rainfall. Hence, the results of this evaluation are based on continuous simulation of runoff and total suspended solids (TSS), to evaluate performance in terms of fraction of average annual stormwater volume treated and fraction of average annual TSS load removed.

7.2 Schemes Overview

BMPs can provide a wide range of storage capacity. Whenever there is storage some treatment occurs through sedimentation at least. Some devices for which storage might be thought to be minimal, such as sand filters, are able to pond water or create a small backwater, such that there may still be some storage volume associated with the device. Within this chapter, a distinction is made between devices that are primarily flow-limited, that is, for which storage is minimal and for which treatment capacity is governed by the inflow or flow-through capacity, and devices that are storage-limited, that is, devices that inherently store water as part of their fundamental design. In order to regionalize the effectiveness of BMP devices throughout the nation for the purposes of sizing, four continuous simulation procedures were developed as described below:

Scheme 1). Continuous simulations that characterize flow-limited controls, such as screens, filters, and hydrodynamic devices. These controls have minimal or negligible storage. Infiltration devices, such as filter strips, may also be characterized by this scheme, although they are often designed to accept all flows. The scheme is further subdivided into two categories:

Scheme 1a). Continuous simulations that provide a plot of percent of average annual runoff volume captured by a control designed for a certain flow rate. This provides a way to characterize flow-limited controls with minimal and/or negligible storage and no infiltration. The scheme is shown conceptually in Figure 7-1. Bypassed water is assumed to be untreated. Models such as SWMM, spreadsheets, or sometimes simply a frequency analysis of rainfall may serve for this evaluation.

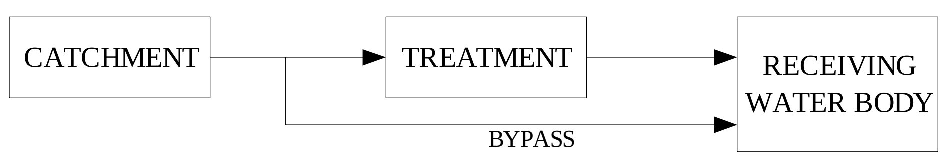

Figure 7-1. Flow-limited BMPs with a bypass (Scheme 1a).

Scheme 1b). Continuous simulations that provide a similar result to scheme 1a for infiltration devices, with minimal storage. Hydrologic losses are by infiltration and evapotranspiration (ET). BMPs that emphasize infiltration and ET and have *minimal storage effects* include BMP/LID controls such as filter strips, porous pavement, and swales without check dams. The results are presented as a performance curve that shows percent control of volume as a function of selected control variables such as infiltration rate and flow path length. The scheme is shown conceptually in Figure 7-2. Spreadsheets are easily used for this analysis, or SWMM when that model is set up to simulate impervious runoff flowing onto pervious overland flow planes.

Scheme 2). Continuous simulations that provide percent control of volume passed through extended dry detention or any *off-line* control (i.e., a control with a planned bypass), that incorporates storage and treatment. The performance is a function of the device volume (expressed herein as inches over the catchment) and of the drawdown time, typically 24 to 72 hours for dry detention devices. Wet ponds, i.e., storage with a permanent pool, may be conceptualized by this scheme if their active storage is drawn down within the specified drawdown time. Runoff captured by these off-line devices is assumed to be "treated" with a known efficiency or, alternatively, the effluent quality characteristics are assumed known, such as a known frequency distribution of effluent event mean concentrations (EMCs). The scheme is shown conceptually in Figure 7-3. Spreadsheets or SWMM can be used for the Scheme 2 analysis.

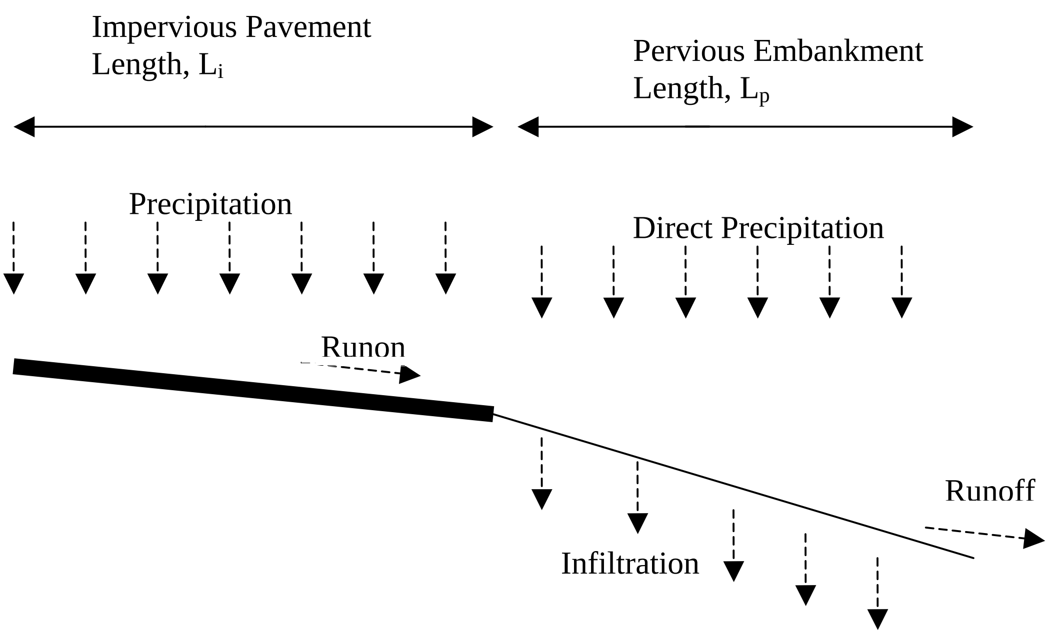

Figure 7-2. Conceptualization of Scheme 1b, for flow-limited infiltration-based BMPs.

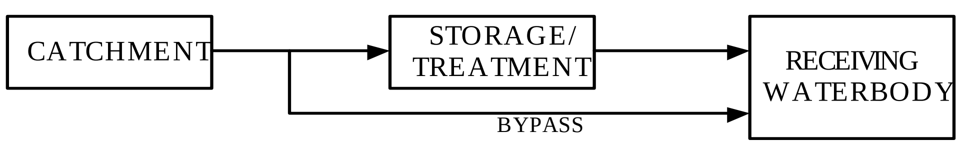

Figure 7-3. Off-line storage-limited BMPs with a bypass (Scheme 2).

Scheme 3). The same continuous simulations as Scheme 2 with the addition of sedimentation for five settling velocity ranges to predict percent total suspended solids (TSS) control as a function of device volume (inches over the catchment) and drawdown time. Scheme

3 applies to o*n-line* storage-treatment devices, such as wetlands, swales, and on-line ponds (no bypass), as illustrated in Figure 7-4. Wet ponds may also fit in this analysis if they are operated on-line. Because TSS performance depends strongly on dynamic detention time, SWMM is used for this analysis in order to simulate sedimentation by settling theory and plug flow.

Figure 7-4. On-line storage-limited BMPs with no bypass (Scheme 3).

Typical BMP/LID facilities falling into each of the four schemes are shown in Table 7-2.

Table 7-2. BMPs as applied to Schemes 1 through 3.

BMP	**Scheme 1a Flow Limited**	**Scheme 1b Infiltration Limited**	**Scheme 2 Storage Limited (off-line with bypass)**	**Scheme 3 Storage Limited (on-line without bypass)**
Tanks/vaults			X	X
Screens	X			
Bar/trash racks	X			
Media/sand/compost filters	X			
Vortex separator	X			
Oil-water separator	X			
Proprietary filtration devices (StormFilter™, etc.)	X			
Extended detention basins			X	X
Retention/detention ponds			X	X
Wetlands			X	X
Wet ponds			X	X
Swales/Bioswales/Filter strips		X		
Infiltration basins		X		
Infiltration trenches		X		
Porous pavement		X		
Dry well				X
Green roofs/LID Elements		X	X	X

All four schemes were evaluated at thirty U.S. locations. Two stations were selected from each of the 15 climate divisions specified by Driscoll et al. (1989, Figure 7-5), discussed in Section 7.4. Complete results are reported in Appendix C, and an additional detailed explanation of methods is provided in the *Final Research Report*, Chapter 10. This chapter focuses on a brief explanation of the methods and how to apply the results.

7.3 The SWMM Model

Several versions of the SWMM program are currently available. These different versions have different simulation capabilities. Two versions were utilized for different purposes throughout the project: version 4.4h (henceforth referred to as SWMM4) and version 5 (SWMM5).

SWMM5, recently developed by the USEPA[1], is a new (2004) and updated version of SWMM, which incorporates a relatively simple and straight-forward object-oriented Graphical User Interface (GUI). The older SWMM4 utilizes discrete blocks within its main program to simulate different hydrologic and hydraulic regimes. It is available from an Oregon State University web site: http://ccee.oregonstate.edu/swmm/. To simulate the rainfall-runoff generation process with SWMM4, several blocks must be linked together and run in series. SWMM4 is therefore not as intuitively simple and easy to use as SWMM5. However, the two versions do not simulate the treatment of water quality in BMPs in exactly the same ways. SWMM5 provides for very convenient user-specified removal functions, applicable to a representation of storage as a continuous-flow, stirred tank reactor (CFSTR). While many storage devices may be approximated as a CFSTR, plug flow with particle settling, which is an option only in SWMM4, may be a more realistic simulation option for extended dry detention and wet detention. Hence, the easier-to-use SWMM5 was used for Scheme 1a simulations, while SWMM4 was used for Scheme 2/3 simulations. A spreadsheet method, discussed below, was used for the infiltration BMPs of Scheme 1b.

7.4 Hydrologic Locations Modeled

For each of the 30 selected locations shown in Figure 7-5, the station name, climate division, elevation, and the distance between the precipitation station and the evaporation station, if any, are listed in Table 7-3; further hydrologic characterization of the 30 stations follows in conjunction with. The evaporation data used for the simulations were obtained from long-term mean monthly values of either pan or Penman evaporation estimates (Farnsworth and Thompson, 1982). In some cases evaporation data were not available for a station selected for precipitation data. In these instances, the nearest station with available evaporation data was used for that location, as indicated in the table. Since the impervious catchment area and the storage configuration (discussed below) were not varied among regions, the hydrologic characterization is a function entirely of precipitation and evaporation differences. Cold climates were modeled as if the entire precipitation record consisted of rainfall. This is based on simulation results that compared frequency analyses (e.g., Figure 7-9) using the entire annual precipitation record vs. using a record from which likely dates with snow and freezing conditions had been expunged, as described in *Final Research Report* Chapter 10. Changes in the frequency analyses were so minor as not to justify the effort required to manually remove precipitation data during below-freezing conditions.

The simulations primarily reflect the interaction between drawdown time or “drain time” (24 or 72 hrs) and storm interevent times, for both quantity control and removal of TSS. Where storm events are typically separated by less than the drain time for a device, performance (percent volume captured, percent TSS removed) is reduced.

[1] www.epa.gov/ednnrmrl/models/swmm/index.htm

Evaporation was simulated from two surfaces: the impervious catchment and ponded water in the BMP (storage or filter strip). To account for differences in evaporation rates between the two surfaces, the evaporation that occurred on the roadway surface was assumed to be equal to that of the raw pan or Penman estimates. The raw pan or Penman methods tend to overestimate the amount of evaporation that occurs off of ponded water. Farnsworth and Thompson (1982) recommend estimation of free water surface evaporation by multiplying the raw pan or Penman mean monthly averages by a coefficient of 0.7. This method was used to estimate evaporation from the storage controls. No infiltration losses were assumed from storage controls (Schemes 2 and 3).

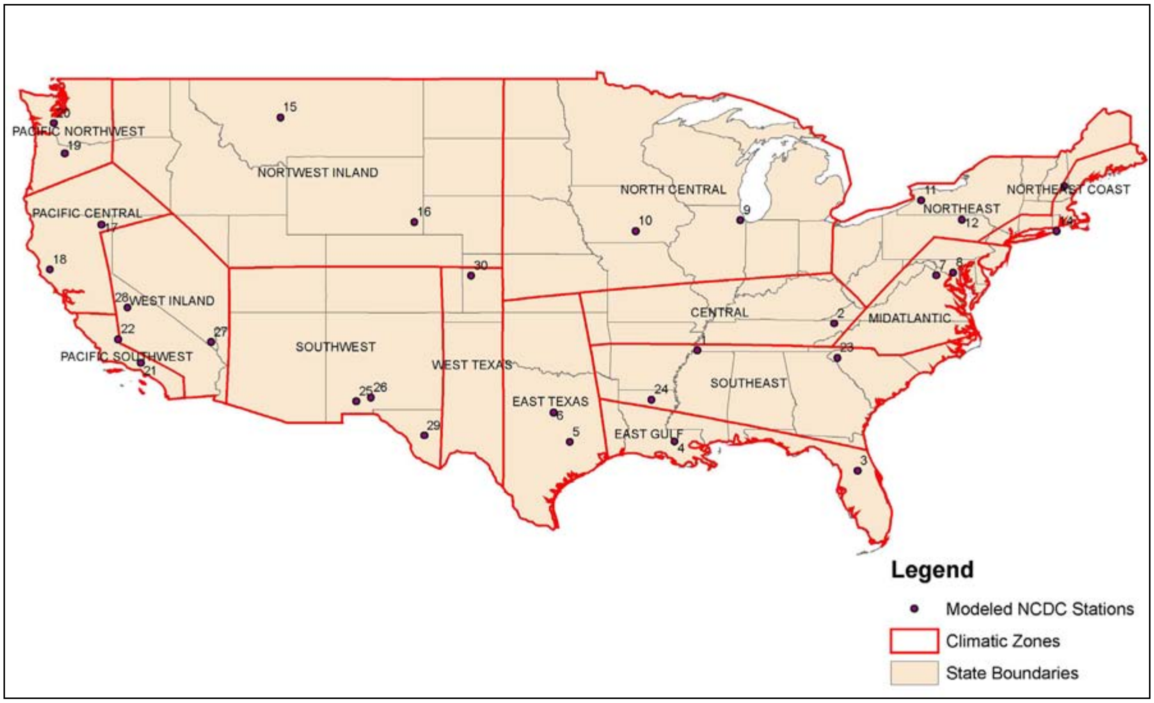

Figure 7-5. Station locations used in the regional hydrologic analysis.

Fifteen-minute rainfall data from the National Climatic Data Center (NCDC) were used for the entire regionalization process because this is the smallest recording interval routinely available around the U.S. Ideally, precipitation data with a smaller time step, one with greater similarity to the residence time of water on the impervious surface, should be used, but the 15-min. data are the best available option. It is preferable to have a resolution of 0.01 in. for these data. Unfortunately, most 15-min. data are only for the coarser resolution of 0.10 in., meaning that the lowest intensity encountered is 0.1 in. in 1/4 hour or 0.4 in./hr. This coarse resolution affected Scheme 1a and 1b results for flow-limited devices for which damping of runoff rates is not afforded by storage. A smoothing scheme was used for low flow rates to avoid a step-function response reflecting 0.1-in. rainfall pulses.

It is customary to use a simulation time step in SWMM that is smaller than that of the residence time of water on the impervious surface. Hence, a computational time step equal to 1 minute was used for Scheme 1a, since these simulations ran quickly (typically 10-20 minutes). Because of the nature of the spreadsheet algorithm (Heaney and Lee, 2005) in which all runoff is assumed to be quickly removed from an impervious surface, a 15-min time step was used for the Scheme 1b simulations. As discussed further below, a 3-min. time step was used for Scheme 2/3 simulations, in order to reduce run times, typically 20-40 min., with the longest lasting about 8 hours for each unit storage data point in the figures provided in Appendix C.

Table 7-3. NCDC stations used for precipitation and evaporation data acquisition.

Map No.[1]	Climate division	Location	Elevation (MSL, ft)	Precip. NWS COOP ID#	Evap. NWS COOP ID#	Distance between stations (miles)
1	Central	Memphis, TN	254	405954	405954	0
2	Central	Rogersville, TN	1355	407884	401094	33
3	East Gulf	Lisbon, FL	68	85076	85076	0
4	East Gulf	LSU Ben Hur Farm, LA	21	165620	165620	0
5	East Texas	Somerville, TX	263	418446	418446	0
6	East Texas	Whitney, TX	574	419715	419715	0
7	Mid-Atlantic	The Plains, VA	530	448396	448084	15
8	Mid-Atlantic	Beltsville, MD	145	180700	180465	16
9	North Central	Chicago, IL	620	111577	111577	0
10	North Central	Columbia, IA	950	131724	132203	35
11	Northeast	Wales, NY	1090	308910	301012	17
12	Northeast	Towanda, PA	760	368905	369728	42
13	Northeast Coastal	New Durham, NH	640	275780	176905	46
14	Northeast Coastal	Block Island, RI	110	370896	376698	39
15	Northwest Inland	Millegan, MT	4500	245706	243751	30
16	Northwest Inland	Phillips, WY	4982	487200	481675	37
17	Pacific Central	Alturas, CA	4400	40161	40161	0
18	Pacific Central	Lake Mendocino, CA	670	44689	44689	0
19	Pacific Northwest	Colton, OR	680	351735	357500	33
20	Pacific Northwest	Centralia, WA	185	451277	456114	18
21	Pacific Southwest	Newhall, CA	1243	46162	45114	32
22	Pacific Southwest	Lost Hills, CA	288	45151	45151	0
23	Southeast	Clemson, SC	824	381770	381770	0
24	Southeast	Calhoun, LA	180	161411	161411	0
25	Southwest	Florida, NM	4450	293225	293225	0
26	Southwest	Jornada, NM	4266	294426	294426	0
27	West Inland	Searchlight, NV	3540	267369	264436	45
28	West Inland	Huntington Lake, CA	7020	44176	44176	0
29	West Texas	Mount Locke, TX	6790	416104	416104	0
30	West Texas	Wallace, KS	3260	148535	143153	32

1 - The map number corresponds to the figure numbering scheme used in Appendix C and may be used as a table of contents for those results.

7.5 Hydrologic Characterization of Stations

A statistical analysis of the precipitation record was conducted for all stations simulated. Both SWMM4 and SWMM5 perform a “SYNOP-type” analysis (Driscoll et al., 1989), in which the precipitation time series (or any runoff time series) may be separated into discrete events using a user-specified minimum interevent time (MIT). Six hours is a typical value used for this analysis, even though in this case, the impervious pavement will drain in much less time. The results of Driscoll et al. (1989) for the 15 regions in general are shown in Table 7-5. For consistency with similar analyses, precipitation statistics for the 30 stations simulated in this project, shown in Table 7-4, are also based on MIT = 6 hrs. This would mean that rainfall pulses separated by 6 hours or greater are treated as a separate event, whereas rainfall increments separated by less than 6 hours are aggregated into one event. Rainfall analysis for the period of simulation is provided for total rainfall depth, number of discrete events, average depth per event, average intensity per event, and average event duration. The reader is cautioned not to infer an average annual rainfall depth by dividing the total rainfall depth by the number of years of record due to the frequent occurrence of missing data from the 15-min. rainfall records.

In addition to the rainfall statistics, continuous simulation results for runoff and evaporation off the impervious surface are shown in Table 7-4 as totals for the period of record, based on the SWMM Scheme 2/3 simulations. A significant amount of the evaporation off the pavement occurs due to the assumed 0.05 in. depression storage. Percent runoff is lower (evaporation is higher) for catchments in more arid areas, as expected. The simulation accounts for the possibility that not all water trapped in depression storage has evaporated by the occurrence of the next storm, just as it accounts for the possibility that the storage device will not be completely drained before the occurrence of the next storm.

7.6 Catchment and Control Schematizations

7.6.1 Subcatchment Properties for Schemes 1a, 2, and 3

The screening simulations described in this appendix are intended to serve the purposes of both the “general urban drainage engineer” and the “highway engineer.” It is not possible to simulate a general mix of impervious and pervious surfaces in the tributary catchment since that would involve far too many variables for screening runs. Hence, an impervious catchment that drains to a BMP is simulated. The imperviousness can represent either 100% of a highway surface or the impervious component of a small urban catchment, since runoff is predominantly from the impervious portion. Thus, results can be interpreted as runoff per acre of impervious surface of the catchment. The hypothetical SWMM subcatchment (Schemes 1a, 2, and 3) is sketched in Figure 7-6 and its properties listed in Table 7-6, the same for all continuous simulations. Justifications for each subcatchment property are given below. The filter strip schematization (Scheme 1b) is discussed subsequently.

The 4-acre area of the impervious subcatchment corresponds to about 2000 ft of a 4-lane highway, with each lane assumed to be 12 feet wide, plus two 10-ft outside shoulders, and two 8-ft inside shoulders (3.86 acres exactly). The characteristic width of the subcatchment was representative of a long rectangle to simulate a roadway, corresponding to a short (transverse dimension) but wide (dimension along the length of the highway) overland flow plane. The pavement slope was 2 percent, mimicking the transverse slope of a roadway surface on a flat area. The value of 0.013 used for Manning’s n is a common value used for concrete and asphalt paving (Engman, 1986). The impervious depression storage value of 0.05 inches is characteristic of pavement. The depression storage is recovered between storms through evaporation.

Results are scaled by catchment area; justification for this scaling is provided in Section 7.11.1.

Table 7-4. Hydrologic characterization of 30 U.S. locations.

Map	Location	State	No. of Events	Total Rainfall (in)	Avg. Storm Depth (in)	Avg. Storm Intensity (in/hr)	Avg. Storm Duration (hrs)	Catchment Evaporation (in)*	Runoff to BMP (in)	Percent Runoff	Period Analyzed (yrs)
1	Memphis	TN	602	309.84	0.516	0.088	6.27	37.34	272.50	87.9	16.5
2	Rogersville	TN	1638	684	0.418	0.108	4.692	90.21	593.79	86.8	16.5
3	Lisbon	FL	1307	701.9	0.537	0.188	3.227	74.77	627.13	89.3	16.5
4	LSU Ben Hur Farm	LA	1259	800.3	0.636	0.178	3.893	75.20	725.10	90.6	16.5
5	Somerville	TX	436	227	0.52	0.157	3.667	25.83	201.17	88.6	16.5
6	Whitney	TX	701	426.4	0.608	0.168	4.19	47.78	378.62	88.8	16.5
7	The Plains	VA	1266	550.7	0.435	0.125	4.554	71.96	478.74	86.9	16.5
8	Beltsville	MD	1288	609.2	0.473	0.122	4.854	76.44	532.76	87.5	16.5
9	Chicago	IL	1208	477.98	0.396	0.108	4.401	68.65	409.33	85.6	16.5
10	Columbia	IA	1294	527.9	0.408	0.126	3.93	73.51	454.39	86.1	16.5
11	Wales	NY	1913	549.9	0.288	0.102	3.839	90.18	459.72	83.6	16.5
12	Towanda	PA	1417	497.2	0.351	0.108	4.15	71.93	425.27	85.5	16.5
13	New Durham	PA	1564	670.2	0.428	0.101	5.008	79.88	590.33	88.1	16.5
14	Block Island	RI	472	201.46	0.427	0.079	6.163	27.62	173.85	86.3	16.5
15	Millegan	MT	1195	229	0.192	0.097	2.617	64.46	164.55	71.9	16.5
16	Phillips	SY	864	202.3	0.234	0.111	2.606	47.96	154.35	76.3	16.5
17	Alturas	CA	977	183.3	0.188	0.09	2.821	43.37	139.93	76.3	15.5
18	Lake Mendocino	CA	881	471.1	0.535	0.087	7.21	62.27	408.83	86.8	15.5
19	Colton	OR	2003	648.4	0.324	0.087	5.019	85.23	563.17	86.9	15.5
20	Centralia	WA	1913	624.5	0.327	0.083	5.271	68.45	556.05	89.0	15.5
21	Newhall	CA	361	233.7	0.649	0.102	6.283	24.98	208.73	89.3	15.5
22	Lost Hills	CA	164	45.45	0.278	0.046	6.399	11.02	34.43	75.7	15.5
23	Clemson	SC	1302	767.5	0.59	0.14	5.148	79.80	687.70	89.6	15.5
24	Calhoun	LA	901	615.9	0.684	0.157	4.722	55.72	560.18	91.0	15.5
25	Florida	NM	155	45.9	0.297	0.116	3.227	10.26	35.64	77.6	15.5
26	Jornada	NM	578	148.3	0.257	0.129	2.419	33.20	115.10	77.6	15.5
27	Searchlight	NV	330	107.7	0.325	0.113	3.818	22.40	85.30	79.2	15.5
28	Huntington Lake	CA	719	498.1	1.453	0.514	1.312	38.09	460.01	92.4	15.5
29	Mount Locke	TX	875	276.4	0.316	0.137	2.699	50.05	226.35	81.9	15.5
30	Wallace	KS	694	216.9	0.313	0.127	2.991	41.45	175.45	80.9	15.5

Data based on rainfall analysis and on Scheme 2/3 simulations

Table 7-5. Storm event statistics based on the 15 climate zones located in Figure 7-5.

TABLE 23.1 Typical Values of Individual Storm Event Statistics for 15 Zones of the United States

Rain Zone	Annual No. of Storms		Duration (hours)		Intensity (in./hr)		Volume (inches)		Storm Separation (hours)	
	Avg.	CV	Avg.	CV	Avg.	CV	Avg.	CV	Avg.	CV
Northeast	70	0.13	11.2	0.81	0.067	1.23	0.50	0.95	126	0.94
Northeast, coastal	63	0.12	11.7	0.77	0.071	1.05	0.66	1.03	140	0.87
Mid-Atlantic	62	0.13	10.1	0.84	0.092	1.20	0.64	1.01	143	0.97
Central	68	0.14	9.2	0.85	0.097	1.09	0.62	1.00	133	0.99
North Central	55	0.16	9.5	0.83	0.087	1.20	0.55	1.01	167	1.17
Southeast	65	0.15	8.7	0.92	0.122	1.09	0.75	1.10	136	1.03
East Gulf	68	0.17	6.4	1.05	0.178	1.03	0.80	1.19	130	1.25
East Texas	41	0.22	8.0	0.97	0.137	1.08	0.76	1.18	213	1.28
West Texas	30	0.27	7.4	0.98	0.121	1.13	0.57	1.07	302	1.53
Southwest	20	0.30	7.8	0.88	0.079	1.16	0.37	0.88	473	1.46
West, inland	14	0.38	9.4	0.75	0.055	1.06	0.36	0.87	786	1.54
Pacific Southwest	19	0.36	11.6	0.78	0.054	0.76	0.54	0.98	476	2.09
Northwest, inland	31	0.23	10.4	0.82	0.057	1.20	0.37	0.93	304	1.43
Pacific Central	32	0.25	13.7	0.80	0.048	0.85	0.58	1.05	265	2.00
Pacific Northwest	71	0.15	15.9	0.80	0.035	0.73	0.50	1.09	123	1.50

Source: Driscoll et al., 1989.

Table 7-6. Subcatchment input parameters for SWMM.

Parameter	Value
Area	4 acres
% Imperviousness	100%
Shape (To give characteristic width)	Long rectangle, to simulate a roadway surface draining laterally.
Width	2000 ft
Slope	2 %
Impervious Manning's n	0.013
Impervious Depression Storage	0.05 inches

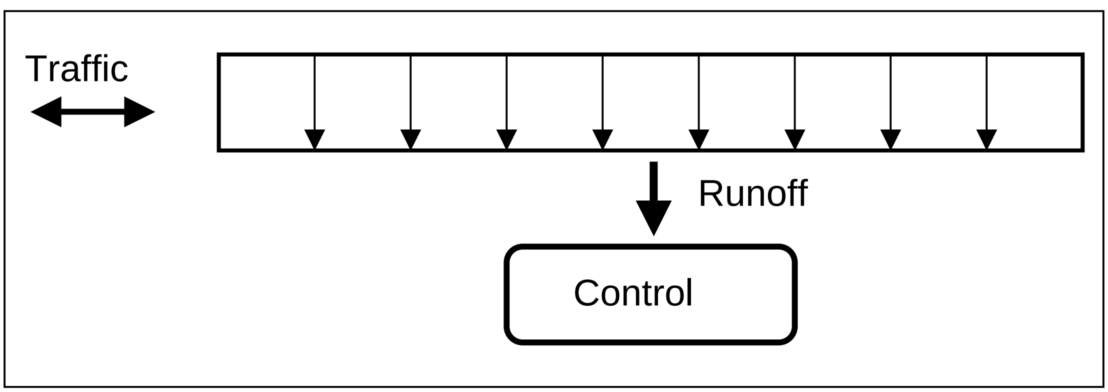

Figure 7-6. Conceptual linear highway catchment draining to control.

7.6.2 Storage Unit Properties

Storage unit properties are only needed for Schemes 2 and 3 listed above because Scheme 1a only analyzes the flow off the subcatchment before the runoff reaches the BMP and Scheme 1b is for infiltration-type BMPs.

Each storage unit volume was determined by multiplying the unit basin size (inches) by the subcatchment area of 4 acres. The total depth of all the storage units is 4 ft, characteristic of many typical stormwater detention facilities (WEF-ASCE, 1998). The surface area is constant as a function of depth. While this vertical-walled shape does not reflect the side slopes of real basins, many real basins have a relatively large base area, surrounded by sloping sides. Hence, the expanding area due to side slopes is typically small compared to the base area. As an example using these assumptions, a unit basin size of 1 inch (1/12 ft) corresponds to a volume of 1/12 ft x 4 ac x 43,560 ft^2/ac = 14,520 ft^3. The surface area, A, is volume/depth = 14,520 ft^3/4 ft = 3,630 ft^2. For purposes of plug flow routing, a length to width ratio of 2:1 is assumed. Hence, the longer dimension of the basin just described would be $L = \sqrt{2A} = 85.2$ ft.

Often, a stormwater control facility is designed for multiple purposes of quantity and quality management. Quantity management – flood control and drainage– usually dictates capture and managed release of as large a runoff volume as possible. “Managed release” in urban areas usually means drawdown within 24 to 72 hours for the design storm in order to maximize hydraulic retention time of the current storm's runoff as well to be ready for subsequent storms. If the outlet releases water too fast, frequent low flows from small storms may pass through unimpeded, with the hydrographs hardly altered and with no quality control because of the short residence time. If the release is too slow, the detention facility will not have full capacity for flood control for the next storm event. Conceptually, this corresponds to a design schematized in Figure 7-7, wherein the “BMP storage” (sometimes the “water quality volume”) corresponds to the design volume for water quality control.

To address the issue of the outlet rating curve – release as a function of depth – sophisticated outlet structures may be designed, in which outflow increases nonlinearly with depth (or with storage volume), as would occur for the storage device of Figure 7-7, in which multiple outlets could be designed to release flows at a higher rate with greater storage depth. Alternatively, a perforated riser, as shown back in Figure 5-33, or similar device would provide for greater outflow at higher depth.

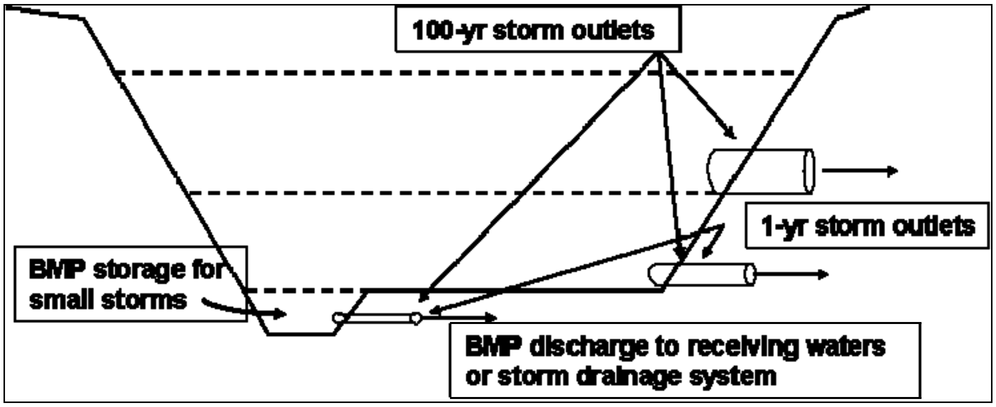

Figure 7-7. Two-stage outlet configuration for water quality and flood control. Reprinted with permission from L.A. Roesner.

For the simulations of this project, two simple rating curves were used. A popular release strategy (WEF-ASCE, 1998; UDFCD, 1999; and CLADPW, 2004) is to empty the top half ("flood or drainage volume") of the storage facility (full to half full) over 1/3 of the drawdown time, and the bottom half during 2/3 of the drawdown time to provide adequate treatment of smaller storms that do not completely fill the storage facility. Hence, the lower "BMP storage" has more time for sedimentation. The first, simpler rating curve is thus a constant outflow for the top half of the storage = twice that for the bottom half of the storage. For example, if the device is designed to drain in 24 hrs when full, the top half will drain in 8 hrs. For the 1-in. basin example presented earlier, the constant top-half outflow rate is thus 14,520 ft^3 / (2 x 8 hr) giving 0.252 cfs, and the constant lower-half outflow rate will be 0.126 cfs.

The second form of rating curve is more representative of a perforated riser. The rating curve (flow, Q, vs. depth, h) can be represented by a power curve of the form,

$$Q = a\, h^b \qquad [7\text{-}1]$$

Where: h = stage
a,b = coefficients; exponent b > 1.

The concave rating curve means low outflow for low depths (the "BMP storage" or "water quality volume") and high outflow for high depths (flood control). An exponent, b = 1.5 fits the outflow characteristics of the perforated riser shown in Figure 5-33 (UDFCD, 1999) and is the value used in the second group of storage simulations presented herein. The coefficient, a, is chosen for each different volume such that the total drain time is either 24 or 72 hrs. Because an outflow of the type given in Equation 7-1 will decrease exponentially, the drain time will theoretically be infinite. This obvious problem is resolved by defining the coefficient a to correspond to release of 95% of the volume, or reducing the depth to 0.05 x 48 in. = 2.4 in. Such a small depth would be "lost" in the reality of an uneven, real, bottom configuration. The rating curves used in the simulations are summarized in Table 7-7. Sensitivity to the choice of rating curve is discussed in Section 7.11.2.

Table 7-7. Rating curve properties used in simulations, for a 24-hr, 1-in. basin example.

Outlet type:	**Perforated Riser**	**Dual Orifice**
Draw Down Times (hrs)	Exponential Drawdown a / b (for 24-hr, 1-in. basin) *where* $Q = ah^b$	Constant Drawdown Bottom Q/Top Q (for 24-hr 1-in. basin) (cfs)
24,72 (Drain time for 95% of total volume)	0.125 / 1.5	-
24,72 (Drain time for 100% of total volume)	-	0.126 / 0.252

Note: Values of Bottom Q and Top Q, and for coefficient a for various basin sizes can be obtained by multiplying the indicated 1-in. values by the basin size in watershed inches. Values of Bottom Q and Top Q, and coefficient a for a 72-hr draw down time can be obtained by dividing the 24-hr values shown in the table by 3.

Two drain times (a.k.a. draw-down times) are used in the simulations: 24 and 72 hrs, covering the range of most municipal guidelines. Results for intermediate times may be obtained by interpolation on the resulting frequency diagrams.

7.6.3 Water Quality Characterization

Sedimentation theory was applied to simulate treatment based upon unit storage and selected drawdown times, which is the prevalent theme in Scheme 3. SWMM4 uses constant or terminal settling velocities as the mechanism to remove discrete particles (Fair et al., 1968; James et al., 2003). (Sedimentation theory and plug flow are not implemented in SWMM5.) Terminal settling velocity occurs when particles accelerate downward through the fluid until their velocity is constant and the drag force of the fluid and the gravitational force exerted on the particle reach equilibrium (Fair et al., 1968). Once particles reach the bottom of the storage unit they are assumed to be removed and cannot re-suspend. Sedimentation theory will also neglect the build up of particles or sludge (“residuals”) generation in all the simulation runs, although the mass is accounted for in the SWMM4 output.

Water and pollutants are routed through the detention basin by assuming plug flow. For this purpose, SWMM4 requires a characteristic travel length for each “plug” of water and is used by the algorithms to determine the settling characteristics of the velocity ranges (James et al., 2003). This travel length is the longer dimension of the basin, described earlier in this subsection.

Two options for settling velocities are available in SWMM4. In the first option, settling velocities are computed for a spherical particle, based on its specific gravity and a drag coefficient that is a function of fluid properties (Reynold’s Number). The second option is simply to provide the settling velocity or settling velocity range directly from settling column tests. The second option is used herein, based on stormwater sampling performed by Liu and Sansalone (Sansalone, J.J. personal communication, 2004) in Baton Rouge, and characteristic of most stormwater particulates. Five different settling velocity ranges are summarized in Table 7-8. When a settling velocity range of pollutants is entered into SWMM4, the program uses the average of the two settling velocities to characterize the range; these averages are also shown in Table 7-8. Sizes greater than 25 μm are assumed to settle completely, on the basis of many trial simulations. If such sizes are present in the influent stream, these can be accounted for in computing the weighted TSS removal or weighted TSS outflow event mean concentration (EMC), as shown in Section 7.12.8.

Table 7-8. TSS settling velocities with approximate idealized particles.

Range[1]	Approximate Size Class[2]	Particle Diameter Range (μm)		Velocity Range (mm/s)		Average Particle Range (μm)	Average Velocity (mm/s)
TSS1	Fine silt	4	6	0.015	0.03	5	0.0225
TSS2	Fine silt	6	8	0.03	0.06	7	0.045
TSS3	Medium Silt	8	10	0.06	0.1	9	0.08
TSS4	Medium Silt	10	15	0.1	0.2	12.5	0.15
TSS5	Coarse Silt	15	25	0.2	0.4	20	0.30

1 - The TSS parameters correspond to SWMM results presented below. Particle size and velocity ranges correspond to TSS data from highway runoff in Baton Rouge, Louisiana, from J. Sansalone (personal communication, 2004).
2 - Characterization from Friedman and Sanders (1978).

7.6.4 Subcatchment Properties for Scheme 1b Infiltration Simulations

Scheme 1b was modeled using the same spreadsheet modeling concept as that of Heaney and Lee (2005) in order to analyze the performance of infiltration BMPs (including LID) and their regional variability. The spreadsheet model was developed on the basis of mass balance relationships, i.e., maintaining an accounting of rainfall, runoff, infiltration, and ET within a time step on the impervious and pervious surfaces. A filter strip (e.g., adjacent to highway pavement) is conceptualized as shown in Figure 7-8, following initial application of the model by Pack (2004). Using the highway catchment described earlier, the impervious catchment is considered as two 2-lane, 42-ft wide subcatchments (W_{imp} = 42 feet) with two adjacent infiltration BMPs, as shown for one side in the figure.

In the spreadsheet continuous simulation, just one side of the two subcatchments is modeled with a unit length in the flow direction. The same input parameters were applied for all of the thirty locations, except for the precipitation and ET data. Depression storage (DS) is assumed to be 0.05 in. for the street pavement (DS_{imp}) and 0.1 in. for the infiltration BMPs (DS_{prv}). (The latter is deemed a conservative – low – value for soil on a highway embankment.) Manning's roughness coefficient (n) for the pervious surface is assumed to be 0.2; it is used to calculate the flow travel time. The modeling results are reported as a function of the width ratio (W_{prv} / W_{imp}) for three typical highway embankment slopes (S_o = 0.05, 0.1, 0.2), based on Caltrans guidelines (http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm, http://www.dot.ca.gov/hq/oppd/hdm/pdf/chp0300.pdf), which recommend slopes of no more than 4 horizontal to 1 vertical (S_o = 0.25). Slopes, S_o, correspond to angle θ_{prv} in Figure 7-8.

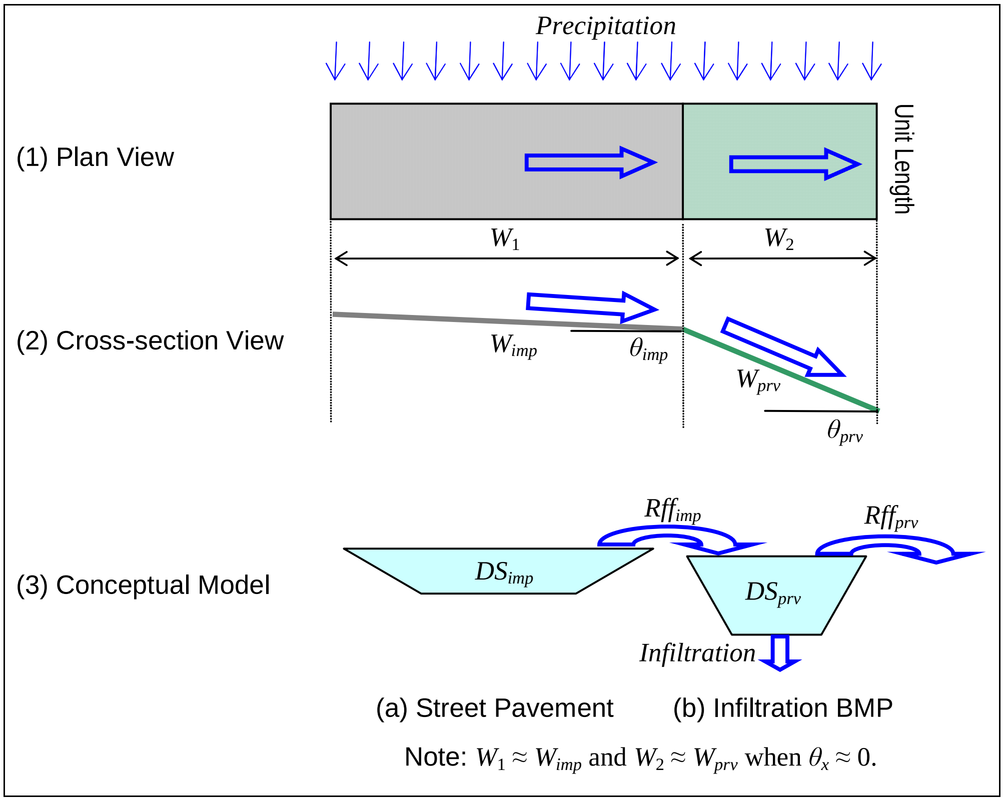

Figure 7-8. Schematic representation of a street and infiltration BMP system.

Each data point in the figures provided in Appendix C for infiltration BMPs represents one model run, with 13 runs for each constant infiltration rate for a total of 52 runs per figure. The 52 runs are performed simultaneously in the Excel spreadsheet through use of a “two-variable data table” feature, in which the results are computed for each combination of four infiltration rates and 13 width ratios. Hence, only three executions of the spreadsheet program are required for each site, one for each slope, and each run only takes seconds. Because the spreadsheet procedure is based on a mass balance, a 15-min time step is used, corresponding to the rainfall input.

Infiltration loss in the spreadsheet model is calculated on the basis of the soil infiltration rate and stormwater flow travel time over the pervious BMP, i.e., residence time, for every precipitation pulse. Flow travel time for the pervious BMP is directly proportional to rainfall intensity. Most of the applied NCDC precipitation data in this project have 0.1-inch of depth resolution for every 15-min recording period, which represents 0.4 in/hr rainfall intensity as a minimum. The actual rainfall intensities, however, are often much smaller than 0.4 in/hr during a real wet period. Therefore, infiltration loss in spreadsheet modeling might be underestimated because of the data resolution of the precipitation input. Hourly data are usually available at a finer resolution (e.g., 0.1 or 0.01 in/hr) to capture the lower rainfall intensities, but the larger temporal resolution would not be directly applicable to the short runoff travel times characteristic of highway catchments. If site specific conditions are known with a better resolution local precipitation data set, a drainage engineer would only need to make the single run applicable to his/her site conditions (in addition to runs for sensitivity analysis, etc.). Similarly, SWMM could be applied for the same purpose, routing the impervious runoff over the filter strip.

7.7 Scheme 1a Results – Flow-Limited Controls

For Scheme 1a, the time period from 1/1980 to 6/2004 was modeled for all 30 locations shown in Figure 7-5. The analysis of these 24-year data sets was completed using SWMM5. After running the continuous simulations for each location, a frequency analysis of the peak flow values generated was performed using SWMM5's "statistics tool." The continuous simulation results (runoff hydrographs) may be separated into runoff events by SWMM using a minimum interevent time (MIT), as described earlier. When the MIT is set to zero, every point on the hydrograph (each 1-min. hydrograph increment) is considered a separate event, and the frequency analysis results in peak flow rate vs. percent of time not exceeded, i.e., a flow-duration curve. Since the flows are equivalent to volumes, on a time step basis, percent annual volume control is obtained. The shape of the graphs generated from this analysis varied between locations, depending upon specific climatic conditions (evaporation and precipitation).

Typical results for flow-limited devices are compared for four locations in Figure 7-9. As can be seen, Lisbon, Florida, in a subtropical climate with frequent thunderstorms (and less frequent tropical cyclones), has a much higher peak flow for a given frequency than do the other locations in the states of Virginia, Rhode Island, and California. The results of Figure 7-9 may be used in two ways. First, if a management criterion (e.g., state or local regulation) says that, say, 90% of the runoff volume must be "controlled" (i.e., pass through the device), then an appropriate design flow may be estimated from such a graph. Second, if a peak flow has already been selected, the percent volume control may be estimated. The portion of flows larger than the control design flow is assumed to be bypassed without treatment, while flows up to the design flow are treated. Any such bypasses are included in the computation of percent volume treated.

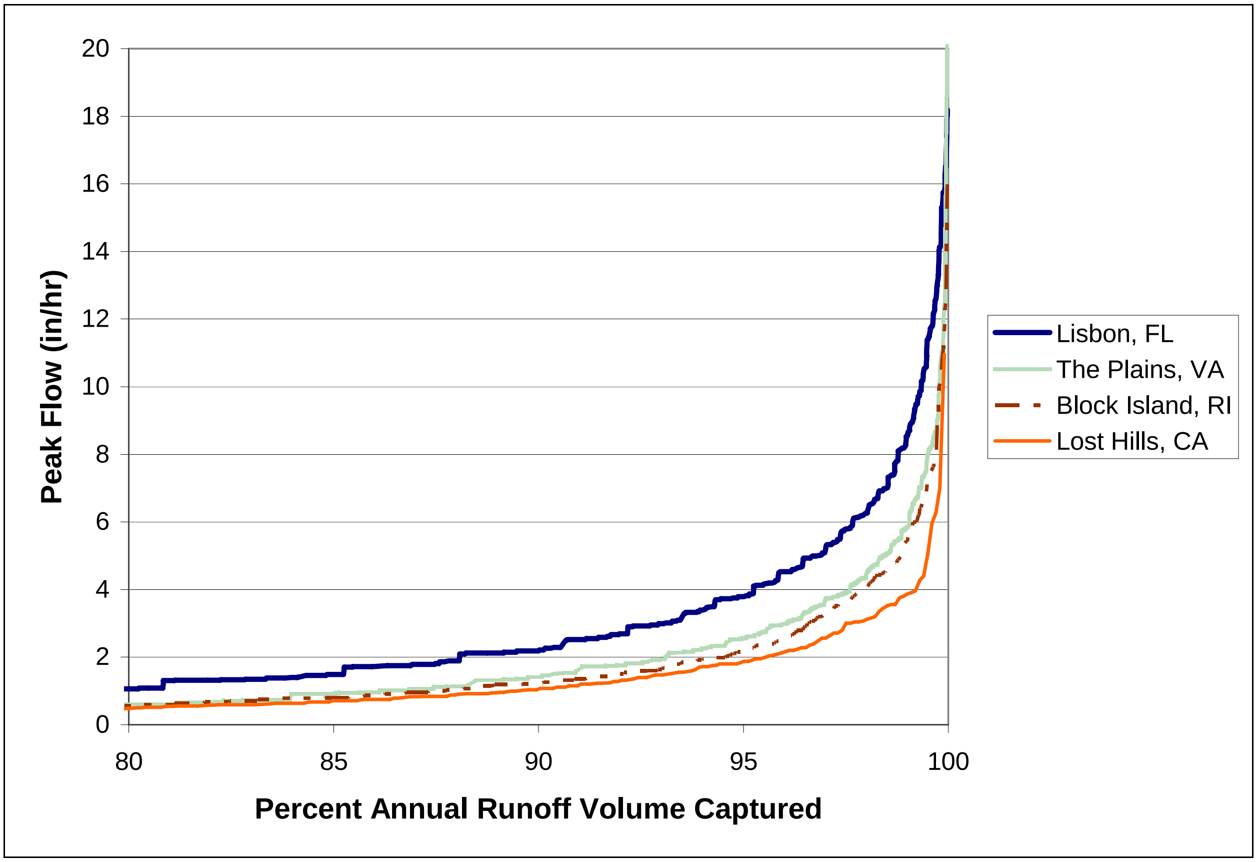

Figure 7-9. Scheme 1a peak flow frequency analysis results for flow-limited devices. Four U.S. locations compared. Only results for control > 80% are shown for clarity.

7.8 Scheme 1b Results – Infiltration-Type Controls

Comparisons of five locations based on volumetric percent capture are presented in Figure 7-10 for a constant infiltration rate of, f = 0.2 in/hr, and three different pervious area embankment slopes. Regional differences of the performance of infiltration BMPs are clearly shown in the modeling results. The figures show both the variation due to regional differences in precipitation and ET and also the effect of embankment slope; it is no surprise that a higher slope decreases infiltration (viz. Figure 7-10). Equally clear is that arid climates have increased infiltration losses because of drier soil conditions in the simulation. (The simulations do not account for possible soil wetting effects that might reduce infiltration during the initial part of a thunderstorm, for instance.)

Rainfall is captured by depression storage, lost to infiltration, and then excess rainfall is released as surface runoff. Figure 7-11 shows the proportions of four hydrologic components for five locations, i.e., DS_{imp}, DS_{prv}, *Infiltration*, and *Runoff*, based on different slopes (S_o) and constant width ratio (W_{prv} / W_{imp}) of 1 and infiltration rate (f) of 0.2 in/hr. Note that depression storage is a surrogate for ET, since in the spreadsheet model all water that is retained by DS is only released through ET. Thus, proportions for depression storage (DS) may be considered as ET losses. The breakdown among runoff, infiltration, ET-pervious and ET-impervious for the 30 stations is shown in Table 7-9. From the table, as well as Figure 7-11, it is clear that drier climates have greater ET and less runoff.

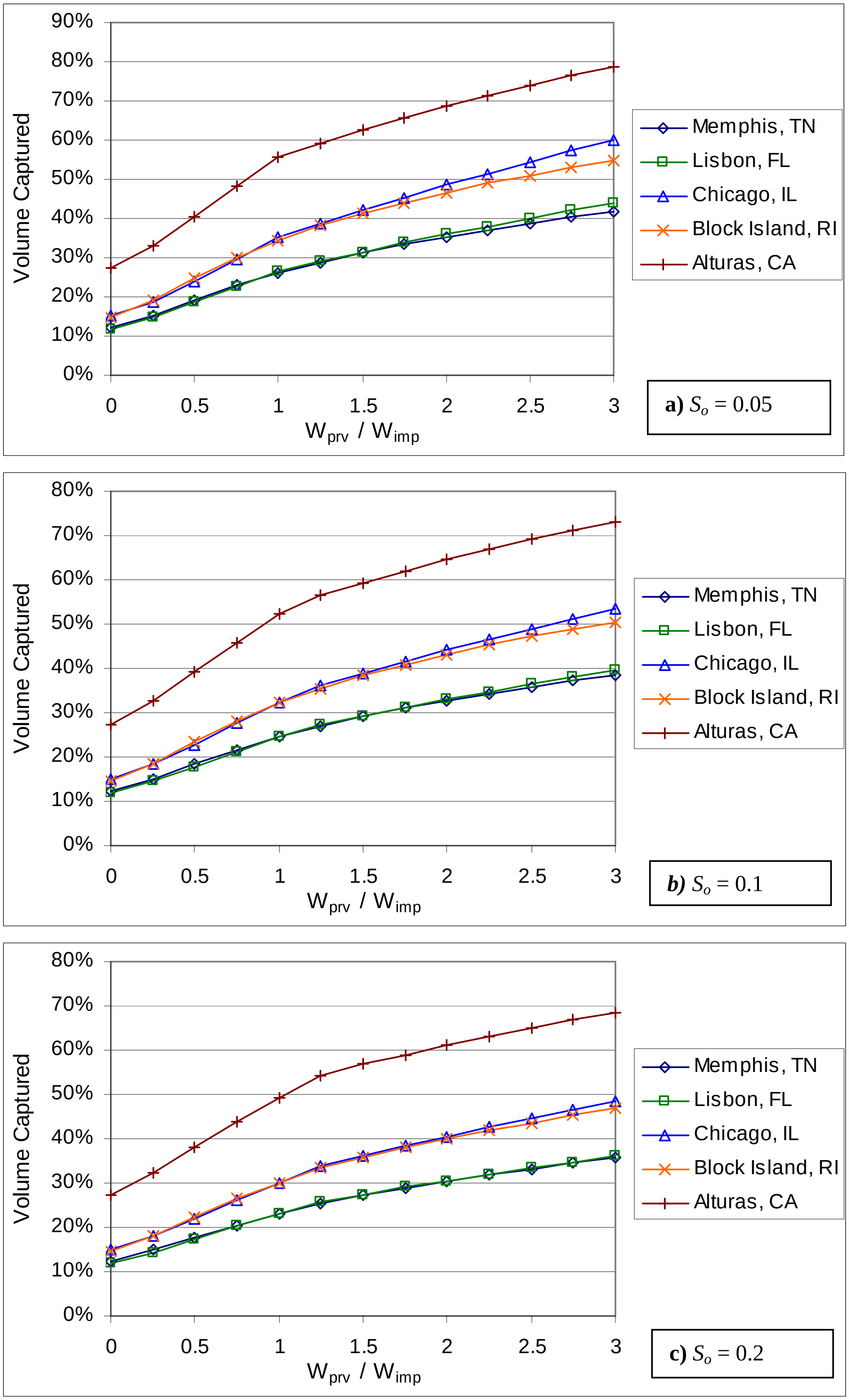

Figure 7-10. Scheme 1b analysis comparing five locations around the U.S.

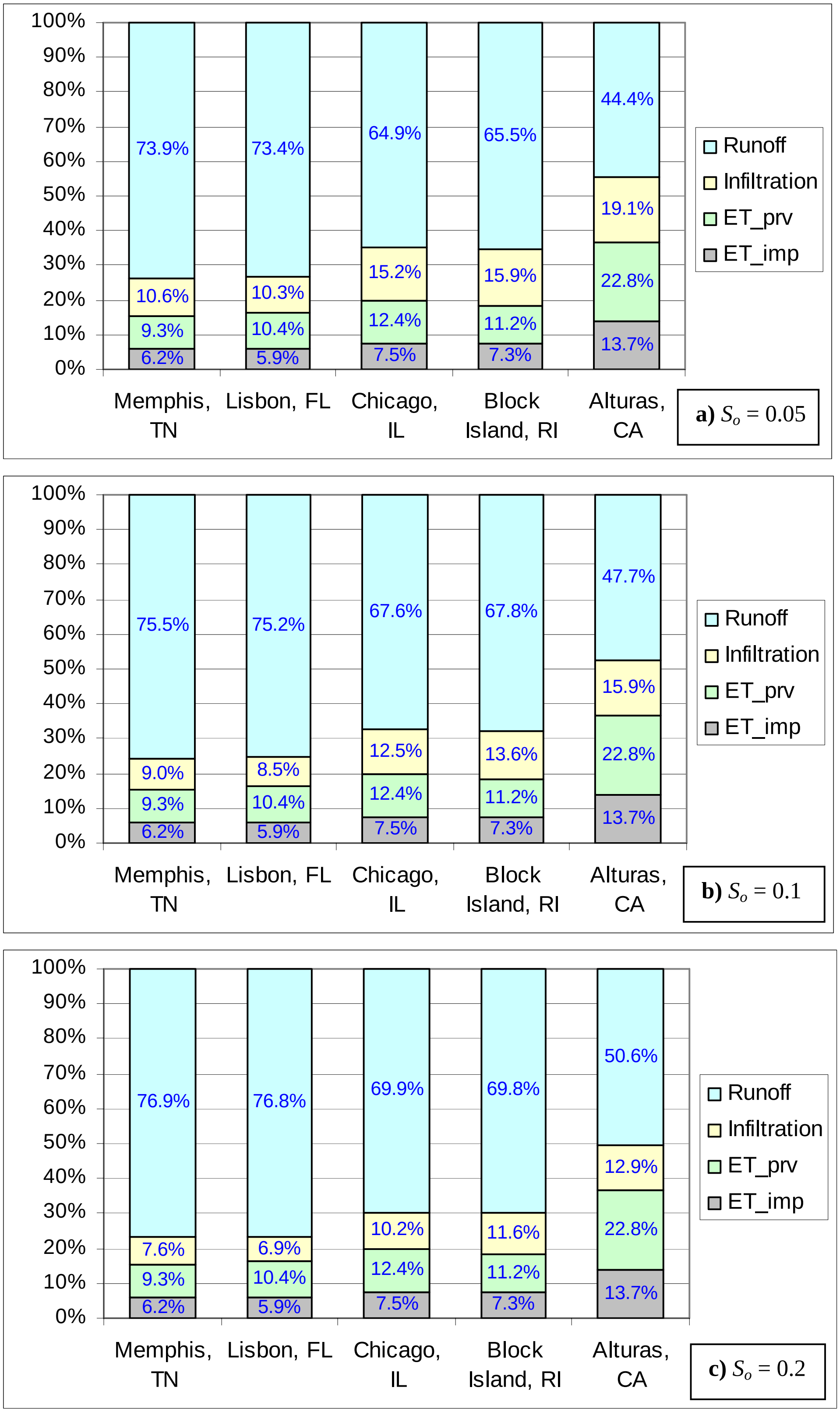

Figure 7-11. Scheme 1b water balance compared among five U.S. locations.

Table 7-9. Water budget for Scheme 1b infiltration-type BMPs for 30 U.S. stations.

Map No.	Runoff	Infilt.	ET_{imp}	ET_{prv}	Map No.	Runoff	Infilt.	ET_{imp}	ET_{prv}
1	75.5%	9.0%	6.2%	9.3%	16	49.1%	14.4%	13.2%	23.3%
2	67.6%	12.7%	7.5%	12.2%	17	47.7%	15.8%	13.7%	22.8%
3	75.2%	8.5%	5.9%	10.4%	18	68.9%	13.6%	7.1%	10.4%
4	77.9%	8.8%	4.9%	8.4%	19	68.5%	13.5%	7.4%	10.6%
5	73.1%	10.0%	6.2%	10.6%	20	70.4%	13.5%	6.6%	9.5%
6	74.7%	9.4%	5.9%	10.0%	21	74.0%	12.5%	5.2%	8.2%
7	69.2%	11.8%	7.2%	11.9%	22	27.4%	43.9%	11.4%	17.3%
8	70.3%	11.6%	6.8%	11.3%	23	73.6%	11.1%	5.8%	9.5%
9	67.6%	12.5%	7.5%	12.4%	24	77.3%	9.8%	4.9%	8.1%
10	69.0%	11.4%	7.4%	12.3%	25	53.6%	13.4%	12.0%	21.0%
11	63.3%	13.2%	9.1%	14.4%	26	53.6%	12.9%	12.0%	21.5%
12	63.3%	14.6%	8.3%	13.8%	27	58.9%	13.3%	10.4%	17.4%
13	69.8%	13.0%	6.5%	10.6%	28	76.8%	12.6%	4.2%	6.4%
14	67.9%	13.6%	7.3%	11.2%	29	61.5%	12.9%	9.3%	16.3%
15	43.4%	15.9%	15.0%	25.6%	30	61.3%	12.2%	9.7%	16.9%

7.9 Scheme 2 Results – Off-Line Storage

Scheme 2 was modeled using the SWMM4 Runoff and Storage/Treatment Blocks for an approximate 15-year time period (see Table 7-4) between 1/1989 and 6/2004, with a time step of 3 minutes. The longer time step was needed in order to reduce the run time, discussed earlier, and the length of intermediate output files; SWMM4 and SWMM5 time series of hydrographs and pollutographs must be less than about 2 Gb (due to an MS Windows XP limitation). However, because storage mitigates short time-increment impacts on hydrographs off the pavement, minimal differences (less than 1 percent) were found in results comparing these simulations at 1-min. and 3-min. time steps. Scheme 2 characterizes control of volume and/or TSS load passed through dry detention or any off-line control (i.e., a control with a planned bypass) which has an assumed treatment rate or known effluent quality.

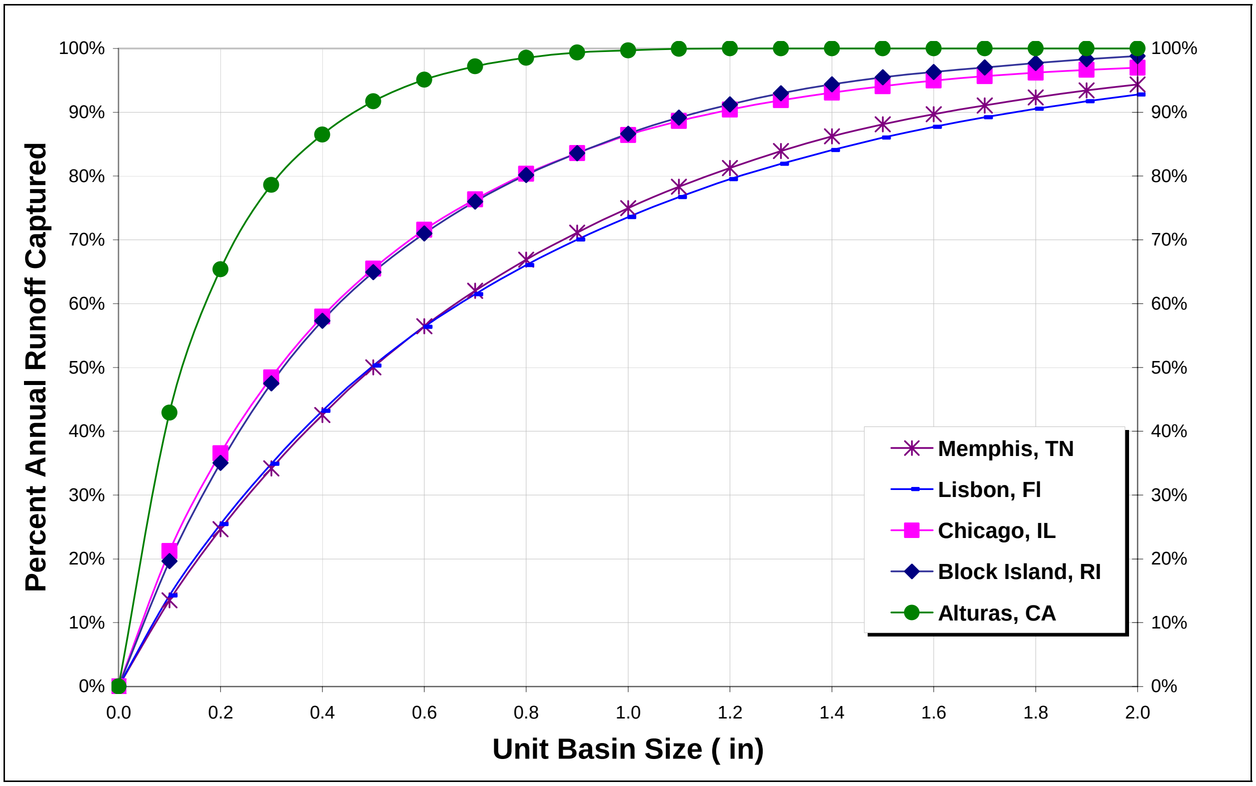

Figure 7-12. Scheme 2 percent runoff capture results for five U.S. locations.

Outlet parameters: 72-hr drawdown time, constant drawdown rates for upper and lower halves of storage (i.e., two-stage outlet).

The Scheme 2 analysis used for characterizing wet weather controls that have a known treatment rate is illustrated in Figure 7-12 for five locations around the country. (In Appendix C, Schemes 2 and 3 are combined in the same figure in order to limit document size.) Regional differences are dramatic. For a given unit basin size, annual percent capture is greatest by far at the more arid Alturas, California site and least for the humid Memphis, Tennessee and Lisbon, Florida sites. Intermediate capture rates are apparent for Block Island, Rhode Island and for Chicago. Regional climatic differences, brought out by the continuous simulations, create the differences evident in Figure 7-12.

7.10 Scheme 3 Results – On-Line Storage

Scheme 3 was modeled using SWMM4 for an approximate 15-year time period between 1/1989 and 6/2004, with a time step of 3 minutes, which is consistent with Scheme 2. The entire results can be found in Appendix C. The figure numbers in Appendix C correspond to the numbers listed in Table 7-3 and Figure 7-5.

TSS removal can be characterized in a basin by a settling velocity distribution similar to the *treatability data* of Figure 7-13. This figure illustrates the five settling velocity distributions that were selected for this simulation, as described earlier in Table 7-8. Notice that the five settling velocity ranges only cover about a third of the TSS settling velocity distribution. Simulations have shown that any settling velocity above 0.4 mm/s, which corresponds to that of a fine sand, settles very quickly, and thus there is no need to model it. If the particle sizes of concern are larger than or equal to fine sand, essentially 100% removal may be assumed.

The percentage of suspended solids removed as a function of the settling velocity distribution is presented in Figure 7-14 for two locations, Lisbon, Florida and Towanda, Pennsylvania. As can be seen there is a large difference in removals between locations. Notice that Lisbon, FL reaches 80% removal of TSS1 (the smallest size range) at 1.42 in. unit basin size, while Towanda, PA reaches 80% removal at 0.73 in. unit basin size. The only model inputs that varied between locations were the evaporation values and the precipitation record used. The evaporation effects are not nearly as important as the precipitation effects for determining the overall characteristics of the model output. This graphical comparison certainly demonstrates that, due to variations in precipitation and ET between different climate divisions, different unit basin sizes are required at different locations in order to achieve the same suspended solids removal percentage. In this example, on-line storage-treatment devices would need to be much larger in Lisbon than in Towanda to achieve the same TSS removal.

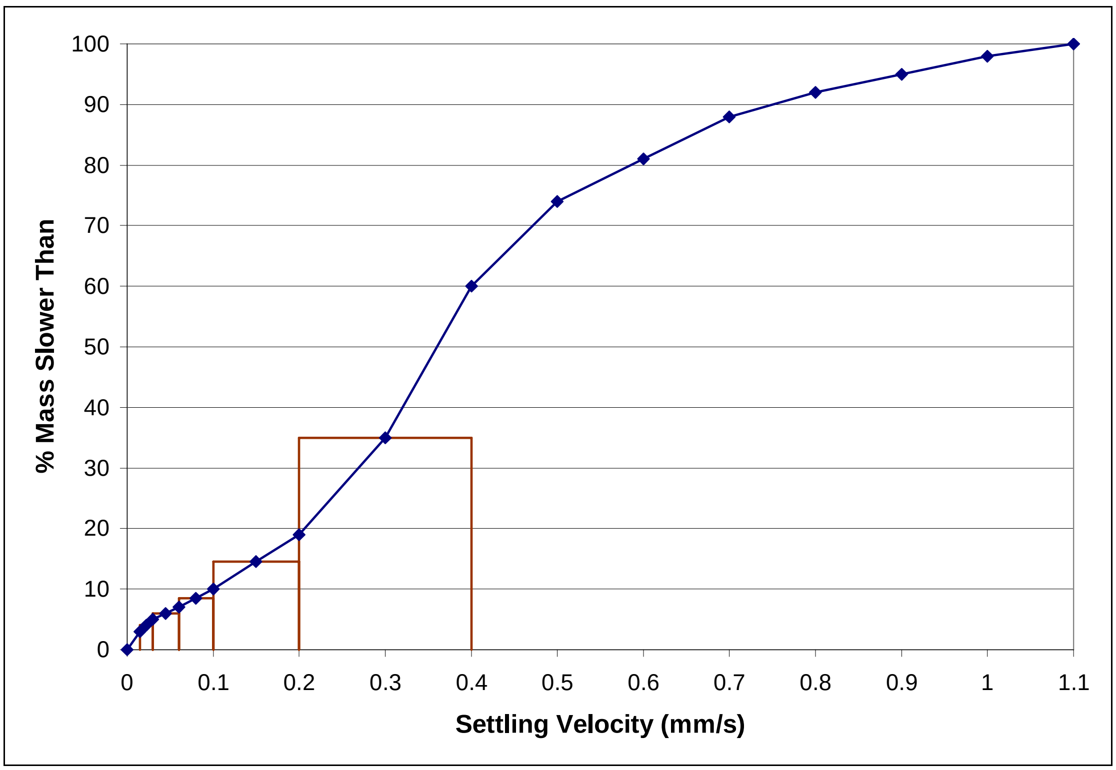

Figure 7-13. Hypothetical TSS settling velocity distribution (treatability data). The five velocity distributions used in the SWMM modeling are shown (Table 7-8).

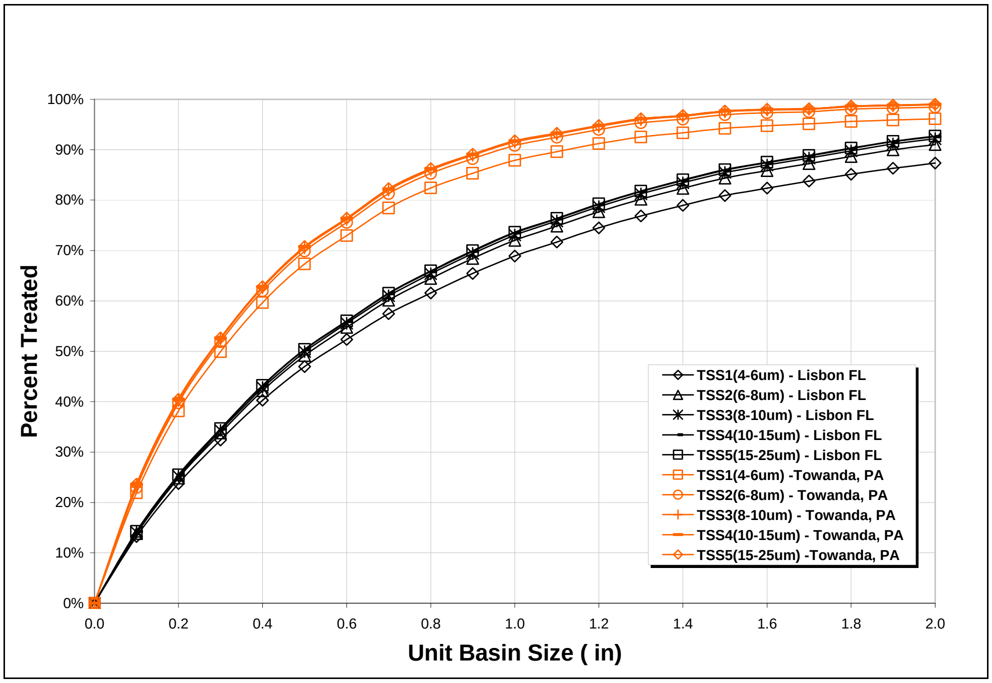

Figure 7-14. Scheme 3 results of TSS treated at two U.S. locations.

Comparing Lisbon, FL (lower curves) and Towanda, PA (upper curves). TSS ranges are defined in Table 7-8.

7.11 Sensitivity to Catchment Area and Rating Curve

7.11.1 Scaling of Results by Catchment Area

All frequency results for Schemes 1a, 2, and 3 are normalized by the catchment area, 4 ac for the simulations described herein. That is, peak flows for Scheme 1a are given in terms of in./hr ≈ cfs/ac. Storage values for Schemes 2/3 are given in units of inches over the catchment area. The results for the infiltration runs of Scheme 1b are already inherently normalized through use of the ratio of the pervious drainage length of that of the impervious drainage length.

Sensitivity to catchment area was investigated by comparing frequency results run for areas greater than 4 ac. Results for peak flow frequencies (Wells, 2005) are shown in Figure 7-15, which also illustrates one comparison related to catchment shape. Considering the area effect first, peak flow – frequency relationships were generated for eight square catchments of different sizes. The eight curves overlay each other; area thus has no effect on scaling for peak flows. However, shape does have an influence. A long narrow catchment representative of a highway (with lateral runoff) responds much faster and with higher peak flows than does a square catchment of the same size. The latter will have a longer flow path length, slower response (i.e., time of concentration), and lower peak flows.

Sensitivity to catchment area of volume capture by storage devices (Schemes 2 and 3) is shown in Figure 7-16 for square areas of 4 and 20 acres. Once again, differences are minimal and overwhelmed by differences between regions. TSS results for the largest size range are very similar to flow capture results.

The conclusion is that these screening results may be scaled by catchment area. Sensitivity to this assumption should be checked by the drainage engineer during detailed design.

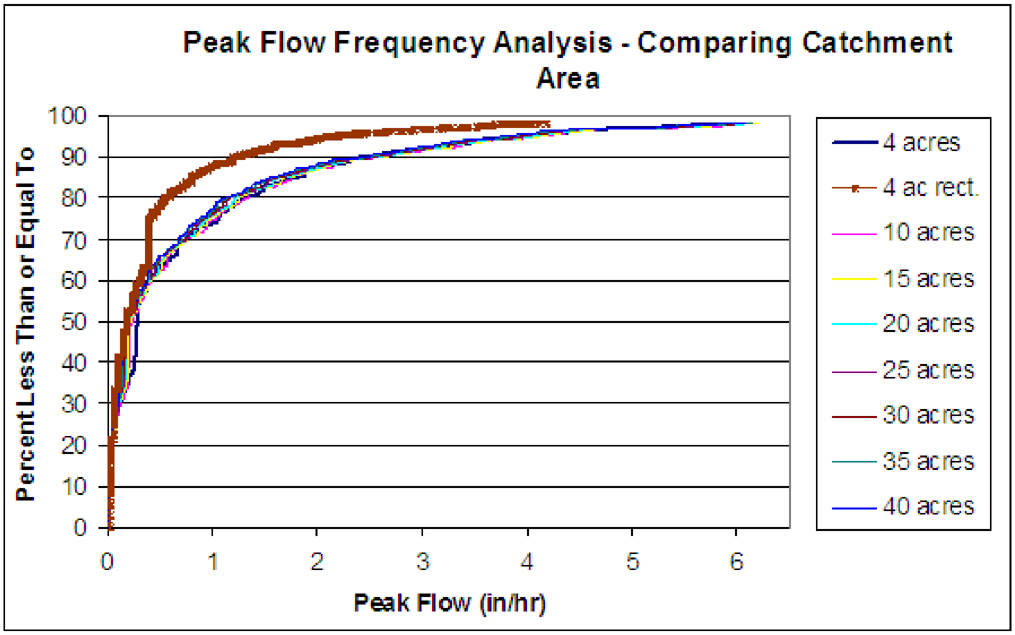

Figure 7-15. Effect of catchment area on peak flow frequencies.

Rainfall data for Chicago (Station 111577).

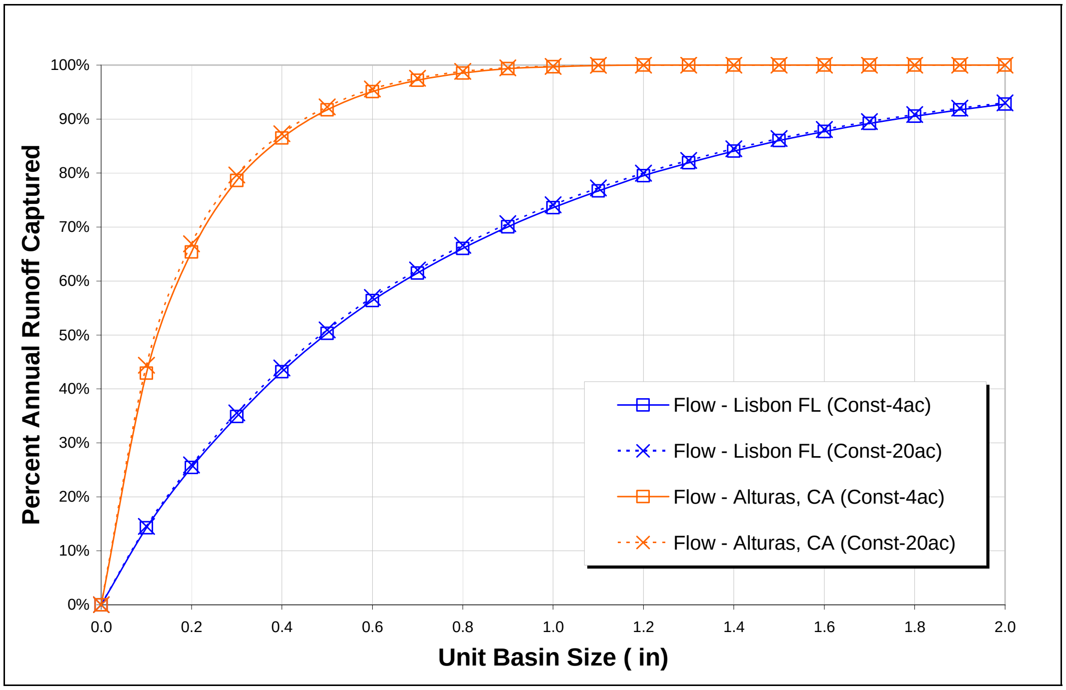

Figure 7-16. Impact of catchment area on performance of storage-limited BMPs (Scheme 2). Locations include Alturas, CA (Station 040161) and Lisbon, FL (Station 085076), 72-hr, 2-stage constant drain time.

7.11.2 Sensitivity to Outlet Rating Curve

Two outlet configurations were presented in Section 7.6.2:

- draw down the top half of the storage at a constant rate in 1/3 of the drain time, and the bottom half at a constant rate half of that for the top half, in 2/3 of the drain time, and
- use a power function rating curve, $Q = ah^b$, representative of a perforated riser outlet device.

The former is referred to as "const" in two figures that follow, and the latter is referred to as "95R," with the "95" meaning draw down to 95% emptiness. A threshold such as this is required since a power function asymptotically approaches zero. Results for runoff and TSS capture are compared in Figure 7-17 and Figure 7-18, respectively.

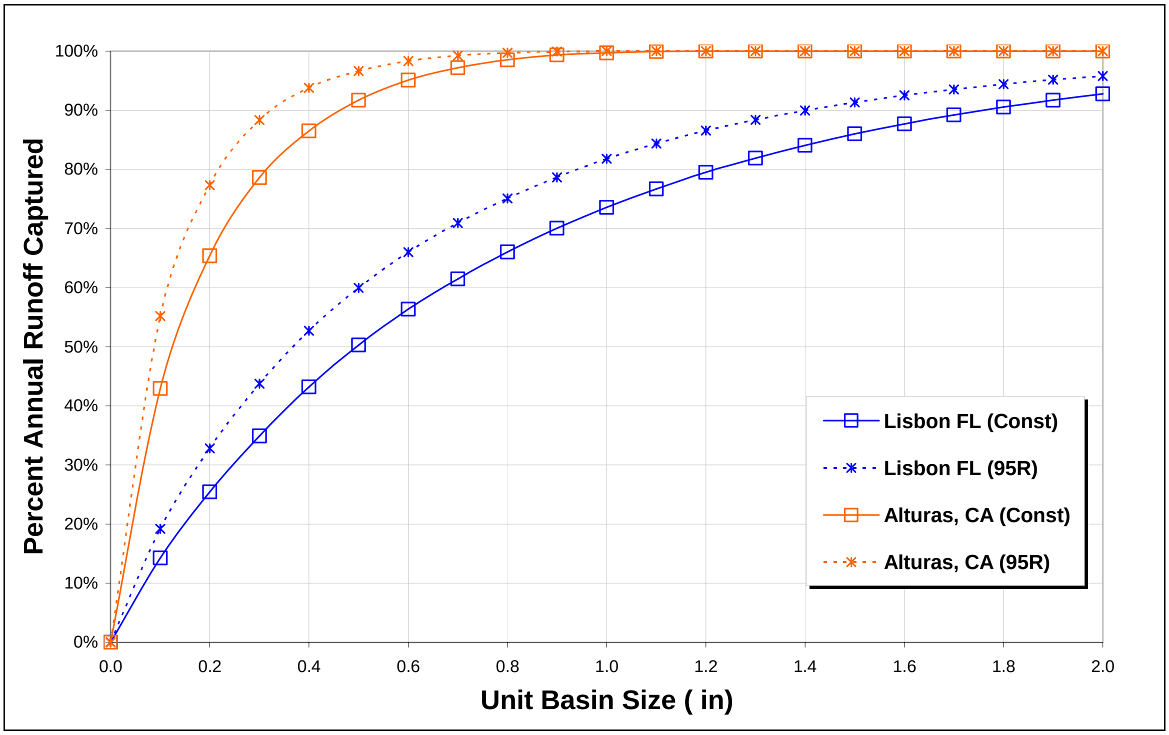

Figure 7-17. Runoff capture results for two different outlet configurations.

Locations include Alturas, CA,(Station 040161) and Lisbon, FL (Station 085076)

72 - hour percent runoff capture (Scheme 2).

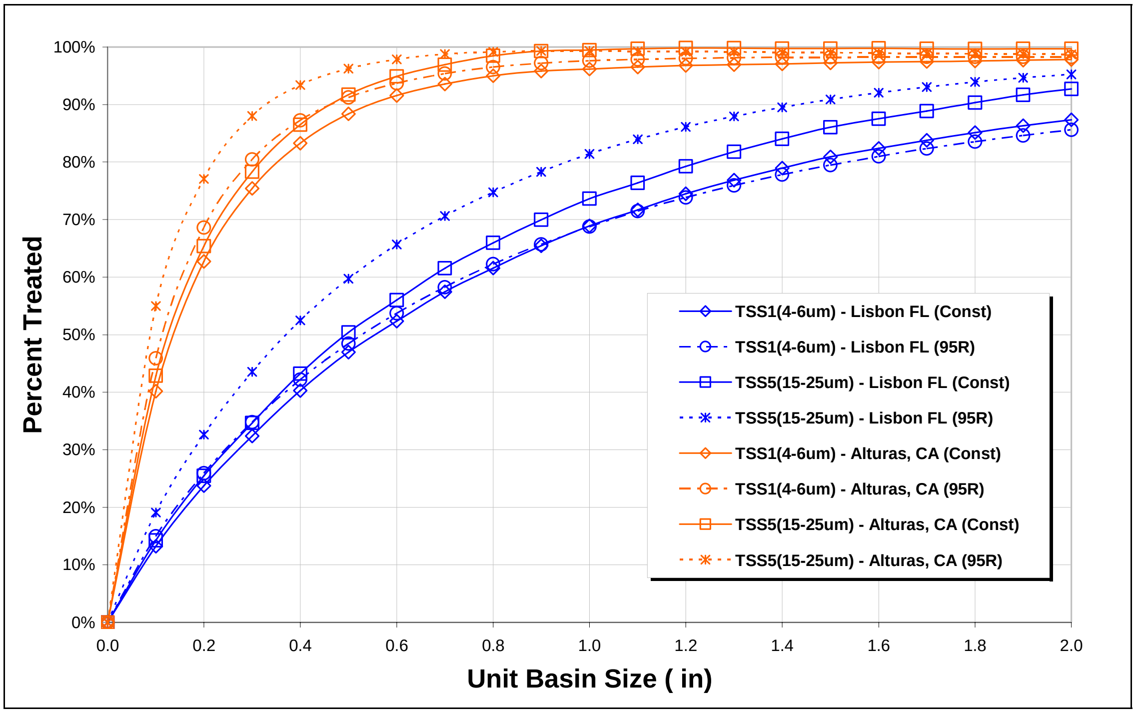

Figure 7-18. Particle capture results for two for two different outlet configurations.

Locations include Alturas, CA,(Station 040161) and Lisbon, FL (Station 085076).

72-hr TSS1 (smallest size) and TSS5 (largest size) removal (Scheme 3), comparing constant and exponential drawdown.

Both figures indicate that performance (runoff captured and TSS removed) is less for the "const" drawdown scheme. For runoff control, it may be seen in Figure 7-17 that the power function-based outflow rate (95R) captures approximately 10% greater volume than the 1/3 – 2/3 constant outflow rate (Const) for unit basin sizes near the knee of the curve. The reason is that the total basin empties somewhat faster when outflow is proportional to head (95R) than if the outflow is constant because storage is created within the facility more quickly. At the same time, TSS removal results (Figure 7-18) are also better with the 95R scheme because the shallower storage depths are emptied very slowly when outflow is proportional to hydraulic head with the exponential stage-discharge equation. Hence, small storm volumes receive greater detention and thus greater TSS removal by sedimentation.

Additional observations from a much broader set of comparisons (not shown) are the following:

- The 24-hr drain time runs show the most variation in performance across outlet designs
- The "const" draw down almost always is the most conservative in terms of % treated (i.e., less % treated for similar basin sizes).
- For any station, the shape of the % treated curve is similar for all outlet types; however, sometimes the curves cross.

- The best performer is site specific; however, the 95R tends to have the best performance at or near the knee of the curves.

Returning to the “const” vs. 95R comparisons, the differences in both cases are not great and are likely subsumed by other site-specific factors that would be included in simulations for individual drainage locations. And for both comparisons, regional results far outweigh sensitivity to the outlet configuration. For the applications that follow, the “const” results are used, since they are more conservative.

7.12 Application of Results

7.12.1 Explanation of Figure Numbering

The process of applying the modeling results involves choosing a scheme (1a, 1b, 2 or 3) that corresponds to a particular BMP, selecting a modeled location, and applying the modeling results. Regional results for all schemes are located in Appendix C. Scheme 1a results are listed in the "flow-based" section (Section C.1) and have figure labels that end in the lower case letter, d. Scheme 1b results are also listed in Section E.1 and have figure labels that end in the lower case letters, a, b or c, depending on the impervious area slope (0.05, 0.1, and 0.2). Scheme 2 results are listed in the “volume-based” section (Section C.2) and have figure labels that end in the lower case letter, a, b, c, or d depending on drain time and tributary area (4 ac/24 hr, 20 ac/24 hr, 4 ac/72 hr, and 20 ac/72hr). Scheme 2 results are plotted in the same graphs as scheme 3 in order to save space.

7.12.2 Selection of Modeled Location and Scheme

The engineer should follow four steps in order to successfully select the results (modeled location) to be used for preliminary screening of a particular BMP of interest:

- **Step 1**. Identify the BMP type and the corresponding scheme (1 to 3) that best represents the BMP of interest as summarized in Table 7-2.
- **Step 2**. Use Figure 7-5 (map of the U.S. climate zones and modeled locations) to determine in which climate zone the BMP of interest will be located.
- **Step 3**. From Figure 7-5, find the closest modeled location (number 1-30) located in the climate zone established in step 2.
- **Step 4.** Once the closest location number in the climate region is selected, look up the modeling results in Appendix C that corresponds to the scheme (1a, 1b, 2, or 3) that best represents the BMP of interest. The modeled location names, National Weather Service (NWS) COOP ID #, and hydrologic characterization can be found in Table 7-3 and Table 7-4.

Comparisons of frequency diagrams computed with different catchment areas ranging from 4 to 40 ac indicate no change in Scheme 1a results with area (Wells, 2005). That is, the scaled frequency vs. peak flows (in/hr ≈ cfs/ac) as in Figure 7-9 do not change as area increases. Hence, peak flows may be scaled by area, as shown in Section 7.12.4. Scheme 2 results for storage size vs. percent capture are somewhat sensitive at low normalized storage values (inches = 12 x ac-ft/ac), with runoff from larger areas (up to 40 ac) captured more efficiently (i.e., having a higher annual volume captured) than for smaller areas. Runoff capture efficiencies are about the same for storages greater than about 0.2 in. For Scheme 3, TSS capture efficiencies are almost the same for all catchment sizes, with a maximum absolute difference of about 10% lower capture efficiency at a unit basin size of 0.8 in for the smallest size range (TSS1) and for a catchment of 4 ac (vs. 40 ac). For the applications that follow, the conservative approach (i.e., performance may be underestimated) of assuming basin sizes are directly scalable by area will be followed. Scalability for Scheme 1b results is provided through the ratio of flow path lengths.

7.12.3 Example BMP and Location Selection

In order to begin selecting a scheme (1 though 3) for a given BMP it is helpful to have a BMP in mind. For example, if space is a limiting factor one might consider a hydrodynamic device over ponds or infiltration BMPs. BMP selection is characterized in the earlier chapters of this Manual. Using these guidelines, the engineer can determine the type of BMP that would most effectively remove pollutants from the stormwater. For example consider three BMPs: an infiltration basin, an on-line detention basin, and a proprietary device. Utilizing Table 7-2, it can be determined that the infiltration basin corresponds to Scheme 1b, the on-line detention basin to Scheme 3 and the proprietary device to Scheme 1a. With this information, the next step would be to determine which location of modeling results to use.

Consider an urban catchment in Atlanta, Georgia. In order to determine which modeling results to use for the initial screening and or sizing, Figure 7-5 is used. From this figure, it can be determined that Atlanta is located in the “Southeast” climate division. Within the “Southeast” climate division the closest simulated station on the map to Atlanta is number 23, which corresponds to Clemson, SC from Table 7-3. In Appendix C, corresponding Figure C.1.12.2d corresponds to Scheme 1a and Figures C.1.12.2a to C.1.12.2c correspond to Scheme 1b, while Figures C.2.12.2.1a to C.2.12.2.2d represent Schemes 2 and 3. Application of the proper schemes for initial screening and/or sizing is demonstrated in the following sections.

The catchment size used in the examples that follows is 5 acres (330 ft by 660 ft) of “equivalent tributary imperviousness.” This means that the catchment might consist of 5 ac of impervious area or mixed land cover with runoff characteristics equivalent to 5 ac of impervious area. The proposed location has a slope of 5 percent for all BMPs used throughout the application of different schemes. For application of infiltration BMPs, infiltrometer tests are assumed to indicate that the soil infiltration rate is approximately 0.6 in/hr.

7.12.4 Scheme 1a – Flow-Limited Device Example

For this example assume that in order to comply with regulations, the City of Atlanta must capture (pass through a BMP) 90% of the runoff and remove 70% of the total suspended solids (TSS). A proprietary device is selected, that has a manufacturer’s claim of 80% removal rate for TSS.

Figure 7-19 illustrates that, in order to capture 90% of the runoff, the selected proprietary device would need to be designed with a capacity of at least 2.4 in/hr. In order to determine the flow capacity of the proprietary device in cubic feet per second (cfs), multiply the design capacity (2.4 in/hr) by the catchment size (5 acres, previously stated). The result is 12 cfs, using the approximation 1 cfs ≈ 1 ac-in./hr.. The first regulatory goal of 90% runoff capture is met by designing our proprietary device to pass 12 cfs. However, it is still uncertain if the TSS removal will be sufficient meet the 70% removal criterion. In order to determine the removal of the proprietary device, multiply the manufacturer’s removal percent (80%) by the percent runoff capture (90%). Carrying out the computation yields a 72% TSS removal (90% x 80%), which is just above the regulatory goal of 70% TSS removal. If for some reason the calculated removal was below the regulatory goal of 70%, then the engineer would either have to increase the flow capacity of the proprietary device and/or use a more efficient proprietary device.

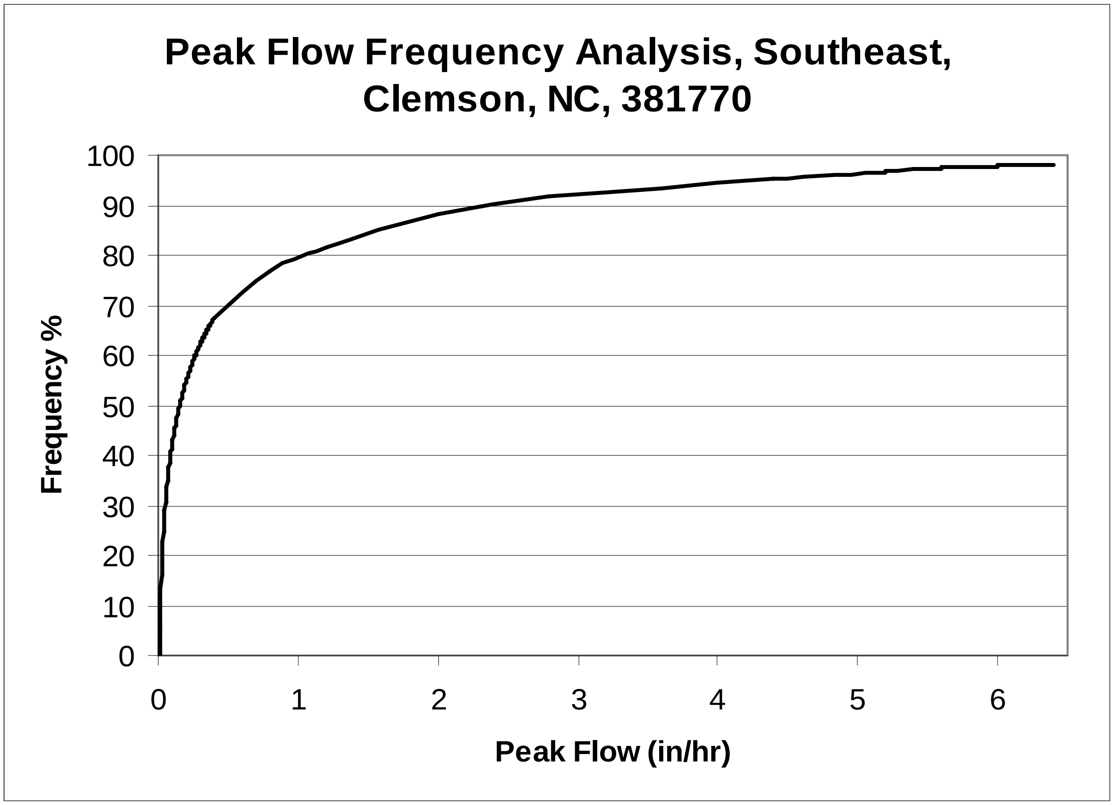

Figure 7-19. Scheme 1a peak flow frequency analysis for Clemson, SC.

Results applied to Atlanta, GA (Figure C.1.12.2-d from Appendix C).

7.12.5 Scheme 1b – Infiltration Device Example

For this example, assume that there is a need to infiltrate 60% of the runoff by use of a broad filter strip or swale. The 5 ac of equivalent tributary imperviousness has an overland flow path length of 330 ft (*Wimp*). For the Atlanta area, Figure C.1.12.2-a fits the site characterization best (5% slope), shown here as Figure 7-20. It is apparent from the figure that with the soil infiltration equal to 0.6 in/hr that the 60% runoff capture goal can be met if the ratio of *Wprv/Wimp* is equal to 1.3. If *Wimp* is equal to 330 ft then *Wprv* would need to be equal to 429 ft (1.3 x 330ft). Assuming the lengths of the catchment and the infiltration area are equal (660 ft) then the infiltration area would need to be 6.5 acres (429 ft x 660 ft x 1ac/43560 ft^2). More simply, if the catchment is made up of 5 ac of impervious area, 6.5 ac of pervious area would be needed to meet the runoff capture goal of 60%. This example also accounts for the effect of ET in reducing the runoff from the pervious area.

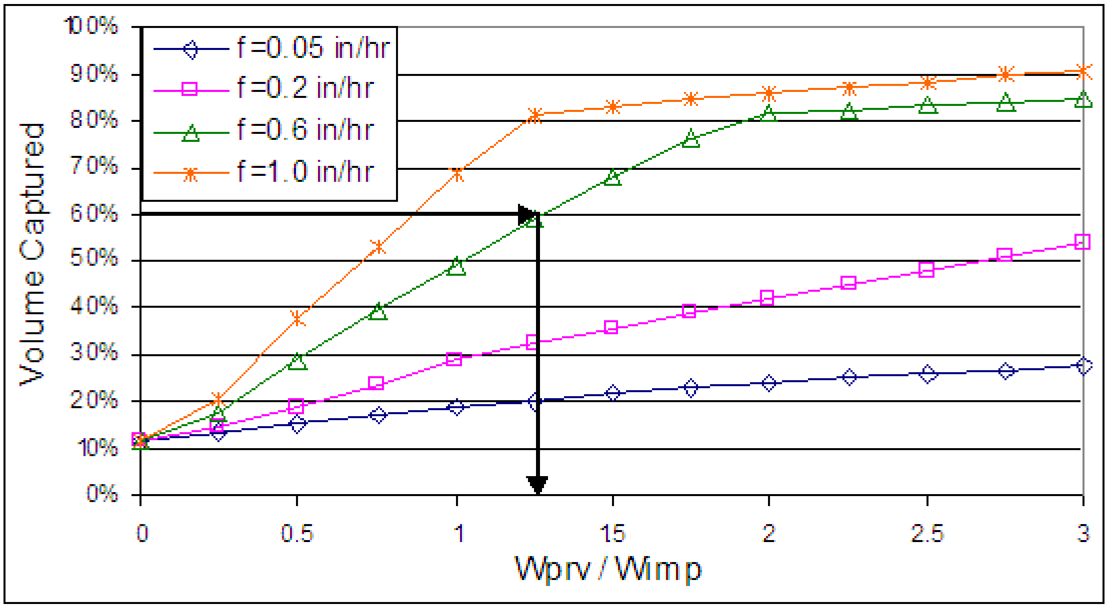

Figure 7-20. Scheme 1b percent volume captured vs. the ratio of *Wprv/Wimp.* Fixed *Wprv* slope, S_o = 0.05. (Figure C.1.12.2-a. from Appendix C).

7.12.6 Scheme 2 – Off-Line Storage Device Example

For this example, again assume there is a need to capture 90% of the runoff and remove 70% of the total suspended solids (TSS). For Scheme 2, all water captured by the device is assumed to have known effluent quality, either in terms of a removal rate or in terms of a known effluent EMC (event mean concentration) distribution. Here, the assumption is that the off-line detention basin has a known TSS removal rate of 80% based on literature values. Assume local regulations require a maximum drain time of 24 hrs.

Since project results in Appendix C are available only for 24 and 72-hr drain times, linear interpolation between charts may be used to obtain values for intermediate times. For instance, if a drain time of 48 hr was required, values midway between the Scheme 2/3 curves for 24 hr and 72 hr could be used. For simplicity, this process is not illustrated herein. Scheme 2/3 charts used for examples will be for the 1/3 – 2/3 constant drawdown evaluations.

Figure C.2.12.2.1-a, shown here as Figure 7-21 illustrates that in order to capture 90% of the runoff the selected off-line detention basin would need to have a unit volume of about 1.0 inches. In order to determine the storage capacity in cubic feet, multiply the design capacity (1.0-inch) by the catchment size (5 acres), resulting in 18,150 ft³. The first regulatory goal of 90% runoff capture is met by designing the offline detention basin to capture this volume. However, it is still uncertain if the TSS removal will be sufficient to meet the 70% removal criterion. In order to determine the removal of the offline detention basin, multiply the known removal percent (80%) by the percent runoff capture (90%). Carrying out the computation yields a 72% TSS removal, which is just above the regulatory goal of 70% TSS removal. If for some reason the calculated removal was below the regulatory goal of 70%, then the engineer would have to increase the size of the offline detention basin. However, it is important to note that a larger basin without an increase in drain time would result in a lower average hydraulic retention time in the basin, which may reduce TSS removal rate below the assumed 80%. The next example illustrates the benefits of using particle settling theory to estimate performance rather than assuming a constant removal rate for all basin sizes and drawdown rates.

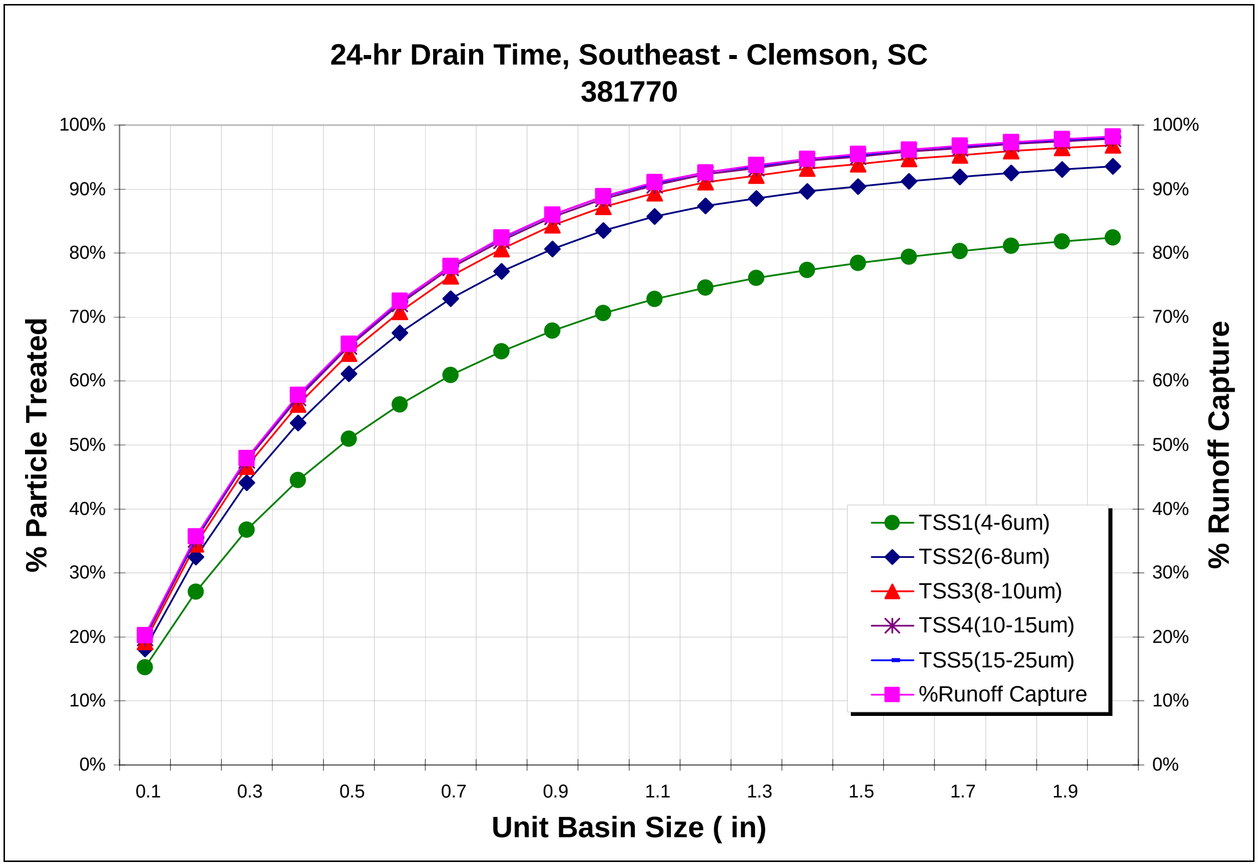

Figure 7-21. Schemes 2 and 3 illustrating runoff capture and TSS removal curves. 24-hr drain time. (Appendix C Figure C.2.12.2.1-a.)

7.12.7 Scheme 3 – On-Line Storage Device Example – Removal Fraction

For this example a requirement to capture 90% of the runoff and remove 70% of the total suspended solids (TSS) is assumed. It is also assumed that local county provisions do not allow stormwater extended dry detention ponds to have standing water for more than 24 hrs due to regulations regarding mosquito propagation. Furthermore, all runoff must pass through the detention pond; hence, performance must account for sedimentation of TSS.

Figure C.2.12.2.1-a, shown here as Figure 7-21, illustrates the results for the 24-hr drain time and associated TSS removal curves, as a function of unit basin size. In order to apply these results one needs a settling velocity distribution characterizing the stormwater TSS similar to that of Figure 7-13, i.e., "treatability data." Such data are shown again, with an expanded horizontal scale, in Figure 7-22. Coupling the results of Figure 7-21 with the hypothetical TSS velocity distribution shown in Figure 7-22, TSS removal can be determined, as shown below.

In order to meet the 90% volume capture associated with the first water quality goal it is apparent from Figure 7-21 that a unit basin size of 1.0 inches would suffice. TSS removal must be calculated in order to determine if a unit basin size of 1.0 inches is sufficient to comply with the second water quality goal (70% TSS removal). TSS removal is determined by first assuming that the percentage of TSS that has a greater settling velocity than 0.4mm/s will settle out completely for the 90% of runoff captured. For this case it can be determined from Figure 7-13 or Figure 7-22 that 40% (100% - 60%) of the TSS from the subcatchment has a settling velocity greater than 0.4 mm/s. The "assumed removal" fraction is the removal of the 40% of TSS greater than 0.4 mm/s that gets removed for 90% of the runoff, yielding a 36 % removal (90% x 40%).

The distribution of the remaining 60% of the settling velocity ranges from Figure 7-13 is enlarged in Figure 7-22. From this figure, the percentage of each segment of the velocity distribution curve can be estimated based on the five velocity ranges. The percentage of each segment under the velocity curve is summarized as the columns titled "Percentage of Velocity Curve Modeled," from Table 7-10 and shown in Figure 7-22. Once the modeled percent removals are known for the size fractions, the iterative process of choosing unit basin sizes until the target total TSS removal is achieved begins. The total TSS removal is computed by weighting the percent removal for each particle size range by the mass fractions from Figure 7-22, given as "% of Velocity Curve Modeled" in Table 7-10 and Table 7-11. Calculations for a first unit volume estimate of 1.0 in. are shown in Table 7-10. "Modeled % Removed" is taken from Figure 7-21. "Calculated % Removed" is determined by the product of columns titled "% of Velocity Curve Modeled" and "Calculated % Removed". The "Total Removal" as shown in Table 7-10 is the sum of the "Calculated TSS Removed" plus the assumed removal of 36% for the "large" particles.

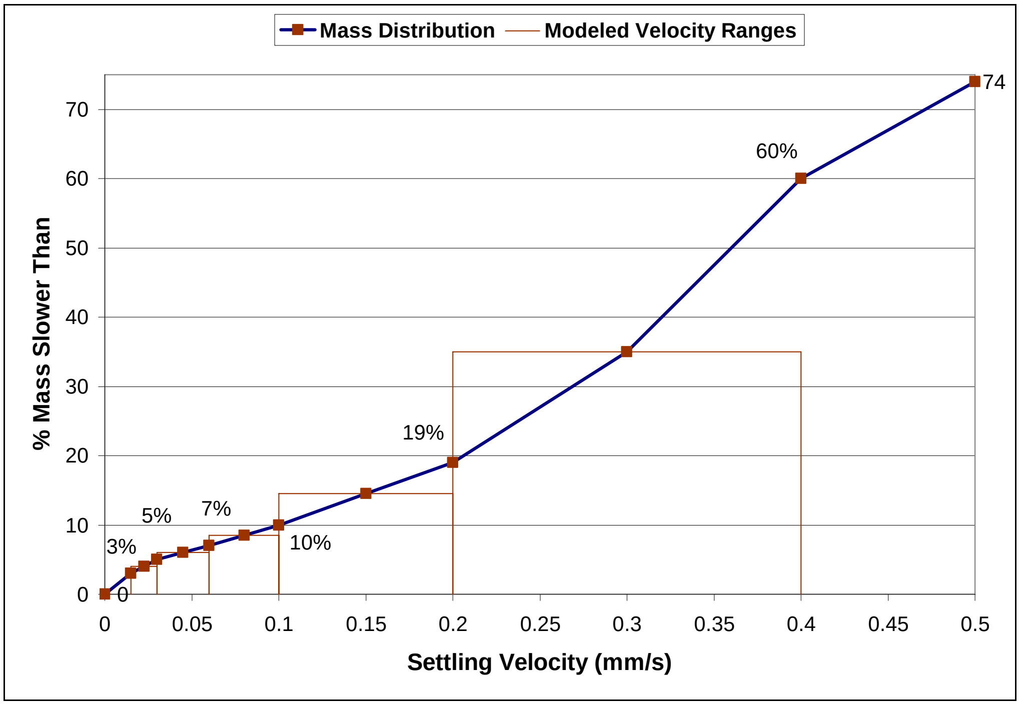

Figure 7-22. Hypothetical settling velocity distribution.

Expanded from Figure 7-13, illustrating the particle size distribution by mass for five simulated average settling velocities of TSS.

Table 7-10. Iteration number one for total TSS removal.

SWMM TSS	Velocity Range (mm/s)		% of Velocity Curve Modeled		Unit Basin Size = 1.0 in. Modeled % Removal	Unit Basin Size = 1.0 in. Calculated TSS Removed
TSS1	0.015	0.03	5% - 3% =	2%	72%	1.44%
TSS2	0.03	0.06	7% - 5% =	2%	85%	1.70%
TSS3	0.06	0.1	10% - 7% =	3%	88%	2.64%
TSS4	0.1	0.2	19% - 10% =	9%	90%	8.10%
TSS5	0.2	0.4	60% - 19% =	41%	90%	36.90%
				Modeled Removal		**50.8%**
				Assumed Removal (>0.4 mm/s)		**36%**
				TOTAL REMOVAL		**86.8%**

It can be seen from Table 7-10 (iteration 1) that a unit basin size of 1.0 inch provides higher (calculated!) removal than the goal of 72%. If this were not the case, the engineer could repeat the computations for a larger basin size (with higher TSS removals).

This example has used the Scheme 3 results to obtain a removal fraction that is applied to the pond screening to obtain results. But recall the discussion of Section 6.8.3 indicating that percent removal is not necessarily a good way to evaluate BMP performance, since “dirtier” inflows inherently show greater BMP removal. Hence, outlet concentrations can also be obtained from the Scheme 3 results, as shown below.

7.12.8 Scheme 3 – On-Line Storage Device Example – Outlet Concentration

For this example, assume a need to capture 90% of the total runoff and that the outlet concentration of the stormwater detention pond should not exceed 25mg/L of TSS. Also assume that the average inflow TSS concentration is 200 mg/L and that local provisions do not allow stormwater extended dry detention basins to have standing water for more than 24 hrs. Furthermore, all runoff must pass through the on-line detention basin; hence, performance must account for sedimentation of TSS. Figure C.2.12.2.1-a, shown here as Figure 7-21, illustrates the results for the 24-hr drain time and associated TSS removal curves, as a function of unit basin size. In order to apply these results one needs the same settling velocity distribution that characterizes the stormwater TSS, namely the treatability data of Figure 7-13 and Figure 7-22 used in the previous example. Coupling the results in Figure 7-21 with the hypothetical TSS settling velocity distribution in Figure 7-22, the average TSS outlet concentration can be determined, as shown below. Since concentration is proportional to mass (for the same unit volume), from the hypothetical settling velocity distribution (Figure 7-22) the concentration of each modeled velocity range (TSS1 to TSS5) can be determined. The concentrations of each velocity range (TSS1 to TSS5) as well as the “Unsettleable” and “Remaining” fractions are shown in Table 7-11 and Table 7-12.

Table 7-11. Estimated TSS outlet concentration for a 1.0-in. unit basin size.

SWMM TSS	**Settling Velocity Range (mm/s)**		**% of Velocity Curve Modeled**		**Conc. of Velocity Range (mg/L)**	**Modeled Removal**	**Conc. Removed (mg/L)**
Unsettleable	0	0.015	3% - 0% =	3%	6	0%	0
TSS1	0.015	0.03	5% - 3% =	2%	4	72%	2.9
TSS2	0.03	0.06	7% - 5% =	2%	4	85%	3.4
TSS3	0.06	0.1	10% - 7% =	3%	6	88%	5.3
TSS4	0.1	0.2	19% - 10% =	9%	18	90%	16.2
TSS5	0.2	0.4	60% - 19% =	41%	82	90%	73.8
Remaining	0.4	1.1	100% - 60%	40%	80	90%	72.0
				Sum:	**200**	**Total Removed**	**173.6**
						OUTLET CONC.	**26.4**
						Sum:	**200.0**

In order to meet the 90% volume capture associated with the first water quality goal it is apparent from Figure 7-21 that a unit basin size of 1.0 inches would suffice. The TSS outlet concentration must be calculated in order to determine if 1.0 inches is sufficient to comply with the second water quality goal (TSS outlet concentration ≤ 25mg/L). The TSS outlet concentration is determined by first assuming that the percentage of TSS that has a settling velocity greater than 0.4 mm/s, called "Remaining" in Table 7-11 and Table 7-12 will settle out completely for 90% of the runoff captured. Another assumption is that the size fraction labeled "Unsettleable" in Table 7-11 and Table 7-12will not be removed because the associated settling velocities are very small. For the "Remaining" case it can be determined from Figure 7-13 that 40% (100% - 60%) of the TSS from the subcatchment has a settling velocity greater than 0.4 mm/s. The "assumed removal" fraction is the removal of the 40% of TSS greater than 0.4 mm/s that gets removed for 90% of the runoff.

Table 7-12. Estimated TSS outlet concentration for a 1.1-in. unit basin size.

SWMM TSS	Settling Velocity Range (mm/s)		% of Velocity Curve Modeled		Conc. Within Velocity Range (mg/L)	Modeled Removal	Conc. Removed (mg/L)
Unsettleable	0	0.015	3% - 0% =	3%	6	0%	0
TSS1	0.015	0.03	5% - 3% =	2%	4	74%	3.0
TSS2	0.03	0.06	7% - 5% =	2%	4	87%	3.5
TSS3	0.06	0.1	10% - 7% =	3%	6	92%	5.5
TSS4	0.1	0.2	19% - 10% =	9%	18	94%	16.9
TSS5	0.2	0.4	60% - 19% =	41%	82	94%	77.1
Remaining	0.4	1.1	100% - 60% =	40%	80	90%	72.0
				Sum:	**200**	**Total Removed**	**178.0**
						OUTLET CONC.	**22.0**
						Sum:	**200.0**

The percentage on a mass basis of each segment of the velocity distribution curve can be estimated based on the five velocity ranges in Figure 7-22. The percentage of each segment under the velocity curve is summarized in Table 7-11 and Table 7-12 as the columns titled "Percentage of Velocity Curve Modeled". Once the modeled removal percentages are known, the remaining concentration of each range can be calculated because the percent mass scales directly with the inflow concentration. Once the concentration of each range is determined, the iterative process of choosing unit basin sizes that correspond to the five different velocity ranges representing removal fractions begins. The weighted TSS "Concentration Removed" is determined by multiplying the "Modeled Removal" by the "Concentration within Velocity Range." Calculations for a first unit volume estimate of 1.0 in. are shown in Table 7-11. The "Concentration within Velocity Range" is calculated by multiplying the "Percent of Velocity Curve Modeled" (from Figure 7-22) by the average inflow concentration equal to 200 mg/L. For example, to find the "Unsettleable" TSS concentration, simply multiply the 3% from the "% of Velocity Curve Modeled" by 200 mg/L, equaling 6 mg/L and so forth. "Modeled %

Removed" is taken from Figure 7-21 and increases with unit basin size. "Concentration Removed" is determined by the product of columns titled "Concentration of Velocity Range" and "Modeled Removal". The "Outlet Concentration" is computed by subtracting the sum of the "Concentration Removed," titled "Total Removed," from the average inflow concentration (200 mg/L). Concentrations can be added and subtracted since they apply to the same water volume; otherwise, loads (mass) would need to be summed.

It can be seen from Table 7-11 (iteration 1) that a unit basin size of 1.0 in. provides a TSS outlet EMC of 26.4 mg/L and is therefore insufficient to meet the goal by a small margin. However, Table 7-12 (iteration 2) shows that a unit basin size of 1.1 inches removes 178 mg/L of TSS and therefore adequately meets the water quality goal, with an outlet concentration of 22 mg/L. Knowing that the catchment is 5 ac the water quality treatment volume needed for the on-line detention pond is the product of 5 ac and 1.1 in., yielding 0.46 ac-ft or 19,965 ft^3.

Note in Table 7-12 that the large "Remaining" particles are assumed to have 90% removal while smaller particles in ranges TSS3, TSS4, and TSS5 have removals higher than 90%, on the basis of the simulation results shown in Figure 7-21. This inconsistency could be removed in two ways: 1) assume the "Remaining" particles also settle with an efficiency of the highest removal for particles smaller than this range, or 2) reduce the earlier ranges to a maximum of 90%, to be conservative. However, there is nothing that prevents almost complete removal of TSS with enough settling time, so the former option may be more reasonable.

7.12.9 Mitigation of Impacts of Imperviousness

The modeling within this chapter is for a 4-ac hypothetical highway catchment (Section 7.6.1), scaled up to 5 ac for the applications of Section 7.12. This catchment is entirely covered by paved impervious area. The entire catchment is modeled as directly connected impervious area (DCIA) for Scheme 1a, 2, and 3 (Figure 7.6), but as indirectly connected impervious area (ICIA) for Scheme 1b (Figure 7.8). In Figure 7-20, a value of $W_{prv}/W_{imp} = 0$ means no pervious area, corresponding to a 100 percent DCIA condition. For this condition, about 12 % of the rainfall is captured only by the depression storage (DS) of DCIA, i.e., depressions in the pavement from which water evaporates between storm events. All W_{prv}/W_{imp} values greater than zero indicate runoff from the now ICIA over some length of pervious surface, with a much greater percentage of runoff captured. For instance, following the example of Section 7.12.5, consider a width (overland flow length) W_{prv} of 429 ft, corresponding to 6.5 ac of pervious area onto which flows the runoff from the 5 ac of runoff from the ICIA. For $W_{prv}/W_{imp} = 429/330 = 6.5/5 = 1.3$, and for an embankment slope of 0.05 and infiltration rate of 0.6 in/hr, only 40% of the rainfall will be discharged beyond the filter strip (bioslope) compared to 88 % when the imperviousness is entirely DCIA.

The same chart can be used for estimating runoff volume reduction by adjusting the mix of DCIA and ICIA, a mitigation principle at the heart of LID, and certainly one of the most effective hydrologic unit operations to be applied to urban and highway runoff. For example, if the DCIA is reduced from 100 % to 40 % for the same design conditions (W_{prv}/W_{imp}, slope, infiltration rate) just used for the pervious area, the mix of DCIA and ICIA can be adjusted to reduce the volume of runoff, as shown in Figure 7-23. In this case, when DCIA is reduced by 60 %, a weighted average percent rainfall volume captured may be computed as shown below:

$$
\begin{aligned}
\textit{Captured Volume } (\%) \quad &= (DCIA \times Capt_{DCIA}) + (ICIA \times Capt_{ICIA}) \\
&= (40\% \times 12\%) + (60\% \times 60\%) = 40.8\%
\end{aligned}
$$

where, *DCIA* = directly connected impervious area (%); *ICIA* = indirectly connected impervious area (%) = 100 - *DCIA*; $Capt_{DCIA}$ = volume captured at the 100 % DCIA condition (%); and $Capt_{ICIA}$ = volume captured at the 100 % ICIA condition (%). Hence, for these particular conditions, by reducing DCIA from 100 % to 40 %, percent runoff is reduced from 88% to 59.2 %. Exactly this kind of analysis could be performed to determine the amount of ICIA and/or increased overland flow length (higher W_{prv}/W_{imp}) would be necessary to mitigate increased runoff volume from the widening of a highway, for instance.

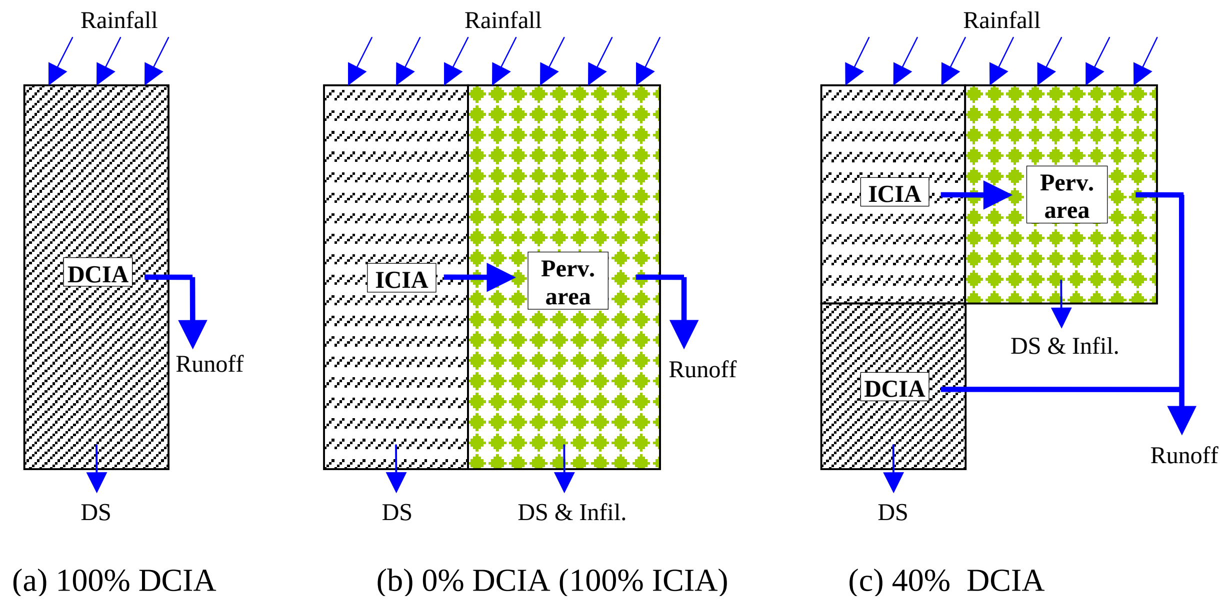

Figure 7-23. Decreasing DCIA to increase runoff capture.

7.13 Regional Screening Summary

The simulations for the 30 locations bear out the assumption (and known empirical fact) that regional hydrologic differences will result in different sizing requirements for storage and infiltration BMPs around the country, i.e., the same unit size will result in different levels of control at different locations. The four Schemes (1a, 1b, 2, 3) may be employed for screening evaluation of BMP sizing depending on the type of BMP to be employed (Table 7-2). When treatability data (particle size distribution percentages) are available, the Scheme 3 results can be combined (percent removal weighted by particle size distribution percentage) to reflect the simulated removal of the actual solids waste stream (Section 7.12.7), or computation of the effluent EMC (Section 7.12.8).

The results shown herein provide *screening* guidelines only, for sizing of BMPs applied to highway and urban runoff control. We recommend site-specific simulation for much more precise design for water quantity and quality control. Nonetheless, the authors are unaware of any more site-specific regional hydrologic analysis of BMP and LID facilities, and the results described in this section and presented in detail in Appendix C should be useful for screening that is more accurate than application of simpler hydrologic procedures (e.g., as described in the *Final Research Report* Chapter 10).

7.14 Flexibility Design / Adaptive Management

The design methodologies described in this document focus attention on selection of specific BMP designs that are anticipated to perform within specified constraints and to achieve established project goals. However, these estimates need to be refined based on the results of actual operation. The concepts of flexible design and adaptive management are important and effective components of implementation and should not be overlooked. These concepts can be quite powerful in situations where effluent quality and downstream hydraulic performance are direct measures of project success, and they allow for changes to be made in system function well after implementation. Continuous simulation of various designs requires the specification of how the system will be operated. It also provides valuable estimates of system behavior across the entire spectrum of flow conditions. Often these design approaches only minimally increase capital costs for a project and can frequently significantly increase the likelihood of achieving project goals.

Flexible design is defined here as having unit operations that can be readily adjusted or modified following construction or installation to achieve variations in system function and performance. Adaptive management is a means for managing these flexible design elements to allow for changes in implementation to be made based on information obtained from monitoring of the effectiveness and performance of a treatment system. Although it is not always possible, ideally adaptive management and flexible design go hand-in-hand and are integral and intentional components of the physical design of a treatment system.

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CHAPTER 8 BMP PERFORMANCE MONITORING AND EVALUATION

Treatment system performance monitoring is conducted by researchers, public entities, and private companies to meet both regulatory and nonregulatory needs. Evaluating the performance of installed stormwater treatment systems can be an in depth and costly process. However, the importance of performance monitoring, particularly for BMPs installed in critical pollutant source areas or upstream of sensitive receiving waters, cannot be overstated.

Several environmental laws exist that mandate implementation of stormwater monitoring programs including:

- The Clean Water Act (CWA) of 1972
- The Endangered Species Act (ESA) of 1973
- Coastal Zone Act Reauthorization Amendments (CZARA) of 1990

Performance monitoring allows for the evaluation of the actual effectiveness of the BMP system, and if carefully designed, allows for the modification of various TSCs to improve performance by adaptive management (see Section 7.14). Monitoring of a stormwater BMP involves 1) determining the scope and objectives, 2) developing a monitoring plan to meet monitoring objectives, 3) implementing the monitoring program, and 4) evaluating and reporting the results. In-depth descriptions of EPA-approved methods for all phases of monitoring, from sampling to data collection, can be found in multiple sources such as the ASCE and EPA Urban Stormwater Monitoring Guidance Manual (2002), Caltrans Stormwater Monitoring Protocol Guidance Manual (Ziegler, et al., 2000), WEF Manual of Practice No. 23 (1998), and USEPA Monitoring Guidance for Determining the Effectiveness of Nonpoint Source Controls (1997). Additional resources relevant to BMP performance monitoring and evaluation are included in Section 8.4.

8.1 Monitoring Program Objectives

A successful monitoring program begins, with a clear identification of monitoring objectives and potential constraints. Stormwater BMP monitoring is initiated to address a broad array of programmatic, management, regulatory, and research goals. Monitoring goals are often focused on achieving water quality objectives (including hydrology/hydraulics and water quality) downstream of the facility. It is important in the planning stage of a monitoring project to quantify the methods that will be used to evaluate attainment of the monitoring goals and objectives.

Before beginning a performance monitoring program, it is important to clearly identify and understand the specific hydrologic and stormwater quality objectives at a particular site, which should have occurred early in the conceptual design phase. This information is useful for determining necessary or potential monitoring and analysis methods. For example, a BMP treatment system with well-defined inlets and outlets, such as a detention pond, would have more monitoring options than a treatment system without well-defined inlets and outlets, such as a bioretention area. Furthermore, a site would require significantly different sampling methods if the pollutants of concern are pathogens or oil and grease, as opposed to heavy metals or nutrients.

8.2 Monitoring Plan Development and Implementation

To support the monitoring objectives, appropriate technologies and current stormwater monitoring knowledge should be incorporated into the monitoring plan where possible.

Developing a monitoring program requires selection of monitoring locations, monitoring frequency, parameters, sampling (e.g., grab or composite, manual or automated) and analytical methods, as well as storm criteria such as size, duration and season. A quality assurance and quality control (QA/QC) plan, which describes the data quality objectives of the monitoring program, and a health and safety plan, should also be developed as part of the monitoring program.

For nonregulatory efforts, or for BMPs designed to reduce runoff volume or rate, the monitoring plan may consist of simple flow monitoring; even visual inspection during runoff events may be sufficient to evaluate performance. Downstream channel or streambank conditions can be visually inspected to assess if erosion, scour, or cut-out has been reduced. To determine the effectiveness of an infiltration system, it may be necessary to monitor influent flow rates, and compare these with overflow data from the system. If there is no flow meter, a weir and pressure transducer combination can be used to monitor flow rates and volumes. Monitoring for infiltration rate requires at a minimum some form of water level detection, such as ultrasonic or pressure related detection.

A fundamental requirement of every successful water quality monitoring program is effective and representative sampling of runoff events at the site. Stormwater sampling is very challenging and is subject to unforeseen circumstances, which may include equipment malfunctions, safety issues, discrepancies between the actual event and forecasts, seasonal runoff variations, and other circumstances. One objective of developing a monitoring plan is to minimize these effects through the use of sound monitoring protocols and strategies during the implementation stage.

Various methods, all with different cost and time structures, can be used for sampling. Grab samples at a specific point in time, are most often collected manually and can be labor intensive due to the broad time scale of runoff events. If samples need to be collected at different times, and over extended time periods, automated samplers could be a more cost-effective option (provided the constituent, due to its nature, does not require collection by grab sampling). To set up an effective automated monitoring program, however, initial equipment setup may be costly. To accurately sample a storm event, automated samplers require a flow measurement device, a flow sensor, and a rain gage. Samples must be collected in appropriate containers (e.g., Teflon® or polyethylene for metals, treated glass for other constituents) using clean sampling techniques. Sampler tubing must be an EPA-approved material.

Once sampling equipment has been installed, two types of sample programming methods can be employed. The most common and cost-effective is flow-weighted composite sampling. This method uses flow data to collect larger sample amounts during high flows, allowing for a more accurate representation of an entire runoff event. A more costly, but more accurate method is discrete, or grab sampling, which consists of collecting samples from discrete time intervals for individual analysis. Analysis of discrete samples provides a more accurate description of the pollution dynamics throughout a storm event. Grab sampling is also required for constituents that transform rapidly, require special preservation, or adhere to bottles (Ziegler, 2000).

The major cost in a monitoring program is the chemical analyses of samples. A comprehensive analytical suite can be costly, especially if several locations are being sampled. For most stormwater treatment system monitoring, composite sampling greatly reduces analytical costs compared to analysis of discrete samples. However, depending on the specific monitoring goals, such as collection of first flush data, detailed intra-event sampling data may be needed. For projects in which monitoring is of limited scope and time, manual grab sampling may be more cost effective because less equipment is needed to implement the program.

8.3 Evaluation and Reporting Results

Reporting of the monitoring results should include information on sampling and analytical methods, in addition to the water quality data and analysis (including an evaluation of data quality). For each site, sample name, sample time and date, rainfall event including any flow rate measurements taken, when the rain started and stopped, rainfall intensity and the total amount of rainfall should be included.

The data should obviously be evaluated and discussed in terms of meeting monitoring objectives. For example, if the primary objective is to assess stormwater quality, a straightforward comparison of pollutant concentrations in stormwater with regulatory criteria may satisfy the reporting requirements. For long-term trend analysis and comparison, a detailed reporting approach should be developed as data become available and trends become apparent.

8.4 Resources for Treatment System Performance Monitoring

A short list of additional resources relevant to BMP performance monitoring and evaluation is given below:

- WERF (2004). *Post-Project Monitoring of BMPs/SUDS to Determine Performance and Whole-Life Costs*. Alexandria, VA.
- ASCE and U.S. EPA (2002). *Urban Stormwater BMP Performance Monitoring – A Guidance Manual for Meeting the National Stormwater BMP Database Requirements*. Prepared by GeoSyntec Consultants and Urban Drainage and Flood Control District [Online] http://www.bmpdatabase.org/docs.html
- Caraco, D. and Claytor, R. (1997). *Stormwater BMP Design Supplement for Cold Climates*. Prepared by the Center for Watershed Protection for the U.S. EPA, http://www.cwp.org/cold-climates.htm.
- Federal Highway Administration (FHWA) (2000). *Stormwater Best Management Practices in an Ultra-Urban Setting: Selection and Monitoring*. Prepared by Tetra-Tech, Inc. and Hagler Bailly Services, Inc. FHWA-EP-00-002, Washington, DC.
- U.S. EPA (1997). *Monitoring Guidance for Determining the Effectiveness of Nonpoint Source Controls*. EPA/841-B-96-004.
- Green, D., Grizzard, T., Randall, C. (1994). "Monitoring of Wetlands, Wet ponds, and Grassed Swales." *Proc Eng Found Conf Stormwater NPDES Related Monitoring Needs*, p 487-513
- NCASI (2000). "Examination of Alternative Statistical Methods for Monitoring BMP Performance." *National Council on Air and Stream Improvement Technical Bulletin,* 805:1-43
- Oswald, G.E., Mattison, R. (1994). "Monitoring the Effectiveness of Structural BMPs." *Proc Eng Found Conf Stormwater NPDES Related Monitoring Needs,* ASCE, Reston, VA. p 514-529
- Schueler, T. (1987). *Controlling Urban Runoff- A Practical Manual for Planning and Designing Urban Best Management Practices*. Metropolitan Washington Council of Governments. Washington, DC, 240 pp

- Spooner, J., Line, D.E. (1993) "Effective Monitoring Strategies for Demonstrating Water Quality Changes from Nonpoint Source Controls on a Watershed Scale." *Water Science and Technology*, 28(3-5): 143-148
- Strecker, E., Mayo, L., Quigley, M. and Howell, J. (2001). *Guidance Manual for Monitoring Highway Runoff Water Quality*. Final report to the Federal Highway Administration. FHWA-EP-01-022
- Strecker, E.W., Quigley, M.M., Urbonas, B.R., Jones, J.E., Clary, J.K. (2001). "Determining Urban Stormwater BMP Effectiveness." *Journal of Water Resources Planning and Management*, 127(3): 144-149
- Stribling, J., Bicknell, J., Cloak, D., Pitt, R. (2002). "Information and Monitoring Needs for Evaluating the Mitigating Effects of BMPs." *Linking Stormwater BMP designs and Performance to Receiving Water Impact Mitigation*, B.R. Urbonas, ed., Proceedings Engineering Foundation Conference, ASCE, Reston, VA, p 334-335
- Urbonas, B.R (1995). "Recommended Parameters to Report with BMP Monitoring Data." *Journal of Water Resources Planning and Management*. 121(1): 23-34
- Young, G.K., Stein, S., Cole, P., Kammer, T., Graziano, F., and Bank, F. (1996). *Evaluation and Management of Highway Runoff Water Quali*ty. FHWA-PD-96-032. Federal Highway Administration, Office of Environment and Planning

CHAPTER 9 CONCLUSIONS AND RECOMMENDATIONS

The selection of unit operations and processes (UOPs) and associated BMPs for treating specific target constituents in highway runoff should be based on past experience, research, and sound scientific and engineering principles. However, to meet ever more stringent water quality management goals and objectives, stormwater treatment is becoming progressively more complex. Significant data gaps exist for the more advanced treatment mechanisms, such as coagulation/flocculation, sustainable filtration, adsorption, ion exchange, precipitation, biological uptake, microbial transformations, and management of residual materials. Pilot studies are needed to start bridging the knowledge gap between UOP theory and field observations of BMP treatment system performance. Results of pilot studies are also needed for parameter estimation and calibration of fate and transport models.

The following research topics have been identified to fill some of the gaps in current stormwater treatment knowledge:

- Development of techniques for accurately measuring and analyzing individual unit operations and processes within TSCs.
- Evaluation of design variables that are related to biochemical and geochemical treatment mechanisms.
- Evaluation of the effectiveness of combination of sedimentation, filtration, and chemical addition for controlling suspended sediment transport.
- Identification and evaluation of accurate and applicable methods for monitoring particle size distribution of suspended sediment concentrations.
- Correlation of heavy metals concentrations to suspended sediment in urban and highway runoff.
- Evaluation of design and performance with respect to particle size distribution in stormwater runoff and associated metals.
- Evaluation of metals speciation under anaerobic and anoxic conditions.
- Evaluation of seasonal effects of plant species on pollutant removal.
- Evaluation of pollutant removal capabilities of native plants.
- Evaluation of the characteristics and effects of short-circuiting, bypass, and overflow.
- Evaluation of the correlation between hydraulic residence time and performance.
- Development of methods or models for estimating the true hydraulic residence time in stormwater ponds.
- Development of alternative methods to optimize detention basin design to maximize treatment.
- Assessment of retrofit options for flood control basins and systems that maximize water quality control while maintaining adequate flood control protection.
- Development of methods for improving/maintaining hydraulic conductivity of infiltration-based TSCs.
- Development of unit treatment models that incorporate advanced treatment mechanisms, such as sorption, ion exchange, biological uptake, and microbial transformations.
- Development of models for simulation of treatment train hydraulics and pollutant removal.

Just as not all pollutants and UOPs have been presented in this document, there are undoubtedly many more relevant research topics that would benefit the stormwater community. Highway BMP design engineers will inevitably need to consult other sources of information and conduct independent research relevant to their particular site conditions, water quality objectives, and personal interests. As the state-of-the-practice advances with additional research and knowledge to varying degrees and complexities, stormwater treatment professionals will increasingly migrate from treatment system designs based on “black box” estimates of performance to designs that focus on achieving pollutant-specific goals through integration of UOPs and BMPs in treatment trains. With this migration, the stormwater engineering community will continue to progress toward sustainable stormwater management solutions.

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