CONTROLLING URBAN RUNOFF:

A PRACTICAL MANUAL FOR PLANNING AND DESIGNING URBAN BMPs

Department of Environmental Programs

METROPOLITAN WASHINGTON COUNCIL OF GOVERNMENTS

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A Practical Manual for Planning and Designing Urban BMPs

by

Thomas R. Schueler

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Metropolitan Washington Council of Governments

prepared for

Washington Metropolitan Water Resources Planning Board

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AGENCY: The Metropolitan Washington Council of Governments is the regional organization of the Washington area's major local governments and their elected officials. COG works towards solutions to such regional problems as energy shortages, traffic congestion, inadequate housing, air and water pollution. The Washington Metropolitan Water Resources Planning Board serves as the area wide wastewater management agency. The Board is involved with regional policy issues related to provision of adequate wastewater treatment, sludge disposal, non-point source pollution control, and municipal water supplies.

REPORT ABSTRACT:

Manual provides detailed guidance for engineers and siteplanners on how to plan and design urban best management practices (BMPs) to remove pollutants and protect stream habitat. Describes water quality and habitat impact in streams that result from uncontrolled watershed development. Contains a simple method for estimating pollutant export from development sites. Presents a series of tools to assist the site designed in selecting the best BMP option for a site. Provides detailed design guidance on seven major urban BMP practices in use in the Washington metropolitan area: extended detention ponds, wet ponds, infiltration basins and trenches, porous pavement, water quality inlets and vegetative practices. Each BMP is reviewed from the standpoint of stormwater management benefits, pollutant removal, physical feasibility, costs, maintenance requirements, and impact to the environment and adjacent communities. A list of recommended design standards that enhance BMP performance is also presented.

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PREFACE

Over eight years ago, the Water Resources Planning Board (WRPB) published the Guidebook For Screening Nonpoint Pollution Strategies. This landmark document (NVPDC,1979) provided some of the first concrete guidance on how to plan and design urban Best Management Practices (BMPs). Since that time, the emerging field of urban runoff control has expanded dramatically. Local research and experience have greatly added to our capability to predict how well a BMP will perform, how it should be designed and how it must be maintained.

This manual was prepared with the goal of describing the state of the art in urban runoff control and focuses on the Washington D.C., area. As with any emerging field, significant gaps still remain in our understanding, and more research and experience must be gathered. Creative and innovative controls need to be tested, and existing techniques need further refinement. Although gaps do remain to be bridged, enough knowledge has been gained to lay out many important principles relating to the design of urban BMPs. This manual is a first approach towards this goal, and it is hoped that the information it contains will be expanded and refined in the coming years.

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INTRODUCTION

In recent years, a growing body of research has shown that urbanization in a watershed can have adverse consequences on streams and receiving waters. These include an increase in flooding, streambank erosion and pollutant export. Historically, management efforts have primarily concentrated on reducing the risk of downstream flooding. The major tool used in this effort has been the dry detention basin that temporarily stores and releases runoff from large storms to reduce peak stormwater discharges. Over 3000 dry detention basins have been constructed in the Washington, D.C. area over the past two decades. A second approach utilized by local governments has been to restrict development along stream floodplains that are susceptible to frequent flooding. Many local governments have also purchased floodplains for public use as stream valley parks.

While both approaches have proven reasonably effective in curtailing flooding problems, they cannot mitigate the adverse impacts urbanization has on stream habitat or increased pollutant export. During the late 1970s, a series of Best Management Practices (BMPs) were developed for urbanizing areas that could remove urban pollutants and, in some cases, protect downstream aquatic life. Most of these practices involved extra detention, retention or infiltration of urban stormwater to enhance pollutant removal and provide additional stormwater management. Initial field testing conducted in the Washington area, and elsewhere in the country, demonstrated that BMPs could serve a dual purpose; controlling nonpoint source pollution from urban areas while providing effective stormwater management.

Since the development of best management practices, a profusion of laws, regulations, and policies have been adopted, at both the local and state level, to encourage or mandate the use of urban BMPs. Recent efforts have caused a dramatic shift in the techniques used to manage stormwater. In a 1982 survey conducted by the Metropolitan Washington Council of Governments (MWCOG, 1984), about 10% of stormwater structures constructed in the region were capable of providing effective pollutant removal; whereas, preliminary results from a similar survey in 1986 indicated that over 50% of stormwater management structures have such capabilities.

Effective implementation of urban BMPs, however, has been mixed. An important factor has been the lack of practical, detailed guidance on how to plan, design and maintain BMPs at the scale of the development site. This manual is an attempt to bridge that gap. It presents an integrated approach toward urban BMP design, that seeks not only to maximize pollutant removal, but also to minimize costs, reduce future maintenance burdens, and to blend facilities into both the natural and human landscape.

The integrated approach requires more thoughtful planning and sophisticated design. Consequently, a wider circle of people are now involved in the process of implementing urban BMPs including planners, engineers, developers, contractors, landscape architects, biologists, hydrologists, homeowner associations and concerned citizens.

This manual attempts to summarize what is currently known about urban BMPs, and has three basic purposes:

1. To enable planners/engineers to define the impacts that are likely to occur as a consequence of developing a site (**Chapter 1**).

2. To enable planners and engineers to rapidly screen available BMP options, and select the one most appropriate for the unique conditions at a particular development site (**Chapter 2**).

3. To review the capabilities and limitations of the various urban best management practices that can be used to mitigate these impacts, with a special emphasis on design considerations that maximize pollutant removal, reduce maintenance requirements and construction costs, and provide environmental amenities (**Chapters 3-9**).

Scope of Manual

The manual summarizes recent local and national research on BMP performance, design and costs, as well as the practical experience gained in urban BMP implementation at the local level.

Due to the wide diversity in existing local stormwater management regulations, BMP design requirements will vary from jurisdiction to jurisdiction. This manual is not intended to substitute for or replace local regulations, nor does it attempt to promote a single kind of urban BMP or strategy. Rather, the basic objective of the manual is to encourage more innovative and practical BMP designs, which not only fulfill local SWM requirements, but are also suited to the unique characteristics of the individual development site.

How to Use the Manual

The manual is oriented towards a diverse audience, and is organized so that each user can rapidly find the guidance they may need.

Chapter 1 begins with a general summary of the sequence of environmental impacts and changes that occur as a result of the development process. Next, the specific water quality problems created by urban runoff are reviewed so planners and engineers can determine which urban pollutants should be the focus of control efforts. Chapter 1 concludes with the presentation of a Simple Method for estimating changes in storm pollutant export from development sites.

Chapter 2 outlines some of the factors planners and engineers need to consider when selecting an urban BMP for a development site. Each of the BMP options for a site can only partially mitigate the quantity and quality impacts of urban runoff. A series of screening tools are presented to guide the designer in choosing a BMP. The first group of screening tools can be used to determine which BMP options are most suitable for the site, given its physical conditions and development characteristics. Subsequent tools detail the stormwater management, pollutant removal and environmental benefits of the practice. The section concludes with an index that indicates where further information used in the final design of the BMP can be found in the manual.

Chapters 3 through 9 review the capabilities and limitations of the seven most commonly used urban best management practices in the region. Particular attention is paid to practical design tips and methods that improve the performance of the BMP options. The BMPs reviewed include:

- Extended detention ponds (Chapter 3)
- Wet ponds (Chapter 4)
- Infiltration trenches (Chapter 5)
- Infiltration basins (Chapter 6)
- Porous pavement (Chapter 7)
- Water quality inlets (Chapter 8)
- Vegetative systems: grassed swales, filter strips, marsh creation, urban forestry, and basin landscaping (Chapter 9)

Each chapter begins with a general summary, followed by a brief review of the major design variations of the practice. Subsequent sections detail:

- What kind of stormwater benefits are provided by the BMP, and how they can be augmented.
- What kind of performance it is likely to have in removing urban pollutants, and how it can be enhanced in the design phase.
- What site conditions prevent or restrict the use of the BMP.
- How much it will cost to construct the BMP.
- What are the routine and non-routine maintenance tasks that must be performed for the BMP to function as intended, and how much will these cost.
- What design and construction techniques are needed to prevent premature failure of the BMP.
- What adverse or positive impacts the BMP will have on local habitat or downstream aquatic life.
- What impacts will the BMP have on the human environment (e.g., safety, recreation, community acceptance).

Each chapter concludes with a summary of design features that should be included in a BMP plan. The designer is encouraged to use these as a checklist during the final design process.

A glossary is included at the end of the manual to define some of the terms used in the emerging field of urban nonpoint source control. The derivation of the Simple Method (Chapter 1) and the Bankfull Flooding Frequency Analysis (Chapter 3) are presented in Appendices A and B, respectively. Finally, a reference list is provided for readers who would like to obtain further information on a particular BMP.

CHAPTER 1: THE IMPACTS OF URBAN RUNOFF

This chapter reviews the diverse impacts that the urbanization process has on streams and other receiving waters. It begins with a summary of the likely changes in water quality and hydrology resulting from uncontrolled stormwater runoff. Next, the specific impacts associated with the pollutants in urban runoff are reviewed. The chapter concludes with the presentation of a Simple Method for estimating storm pollutant export from development sites.

STREAM QUALITY AND THE URBANIZATION PROCESS

Urbanization has a profound influence on stream quality. These impacts are readily seen when a stream in an older urban area is compared to one located in a more natural setting. The following narrative describes the sequence of changes associated with development in a hypothetical small watershed. The pattern presented here is generalized from over two decades of local research and experience.

Changes in Watershed Hydrology

The hydrology of a stream changes in response to initial site clearing and grading. Trees that had intercepted rainfall are felled (Figure 1.1a). Natural depressions which temporarily ponded water are graded to a uniform slope. The thick humus layer of the forest floor that had absorbed rainfall is scraped off or erodes away. Having lost much of its natural storage capacity, the cleared and graded site can no longer prevent rainfall from being rapidly converted to runoff.

The situation worsens after construction is completed (Figure 1.1a). Rooftops, roads, parking lots, sidewalks and driveways make much of the site impervious to rainfall. Unable to percolate into the soil, rainfall is almost completely converted into runoff. The excess runoff becomes too great for the existing drainage system to handle. As a result, the drainage network must be "improved" to direct and convey the runoff away from the site (i.e., by installing culverts, curbs, gutters, storm sewers, or lined channels).

In a typical, moderately developed watershed, the net effect of development is a series of changes to stream hydrology (Figure 1.1b), including:

- Increased peak discharges about two to five times higher than pre-development levels (Leopold, 1968; Anderson, 1970).

- Increased volume of storm runoff produced by each storm, in comparison to pre-development conditions. A moderately developed watershed may produce 50% more runoff volume than a forested watershed during the same storm.

- Decreased time needed for runoff to reach the stream (termed the time of concentration) by as much as 50% (Leopold, 1968), particularly if extensive drainage improvements are made.

- Increased frequency and severity of flooding. A short, intense summer thunderstorm that had only slightly raised water levels in the past now turns the stream into a torrent. In a natural state, a stream experiences bankfull discharges (i.e., runoff entirely fills the stream channel) only about once every two years. In moderately developed watersheds, bankfull discharges may occur as often as three or four times a year.

- Reduced streamflow during prolonged periods of dry weather due to the reduced level of infiltration in the watershed. In smaller, headwater streams, the reduction may be enough to cause a perennial stream to become seasonally dry.

- Greater runoff velocity during storms, due to the combined effect of higher peak discharges, rapid time of concentration, and smoother hydraulic surfaces that occur as a result of development.

Changes in Stream Geometry

The channel of an urbanizing stream must adjust to the new hydrological conditions, and this results in the following responses:

- The primary adjustment to the increased storm flows is through channel widening (Figure 1.1c). Numerous surveys (Robinson, 1976; Fox, 1974; Hammer, 1972) and anecdotal evidence (Ragan and Dietemann, 1976) have shown that most streams widen two to four times their original size if post-development runoff is not effectively controlled. The resulting streambank erosion is severe because most floodplain soils are unconsolidated and highly erodible.

- The elevation of the stream's floodplain must increase to accommodate the higher post-development peak discharge rate (Figure 1.1c). Property and structures which had not previously been subject to flooding now may be at risk.

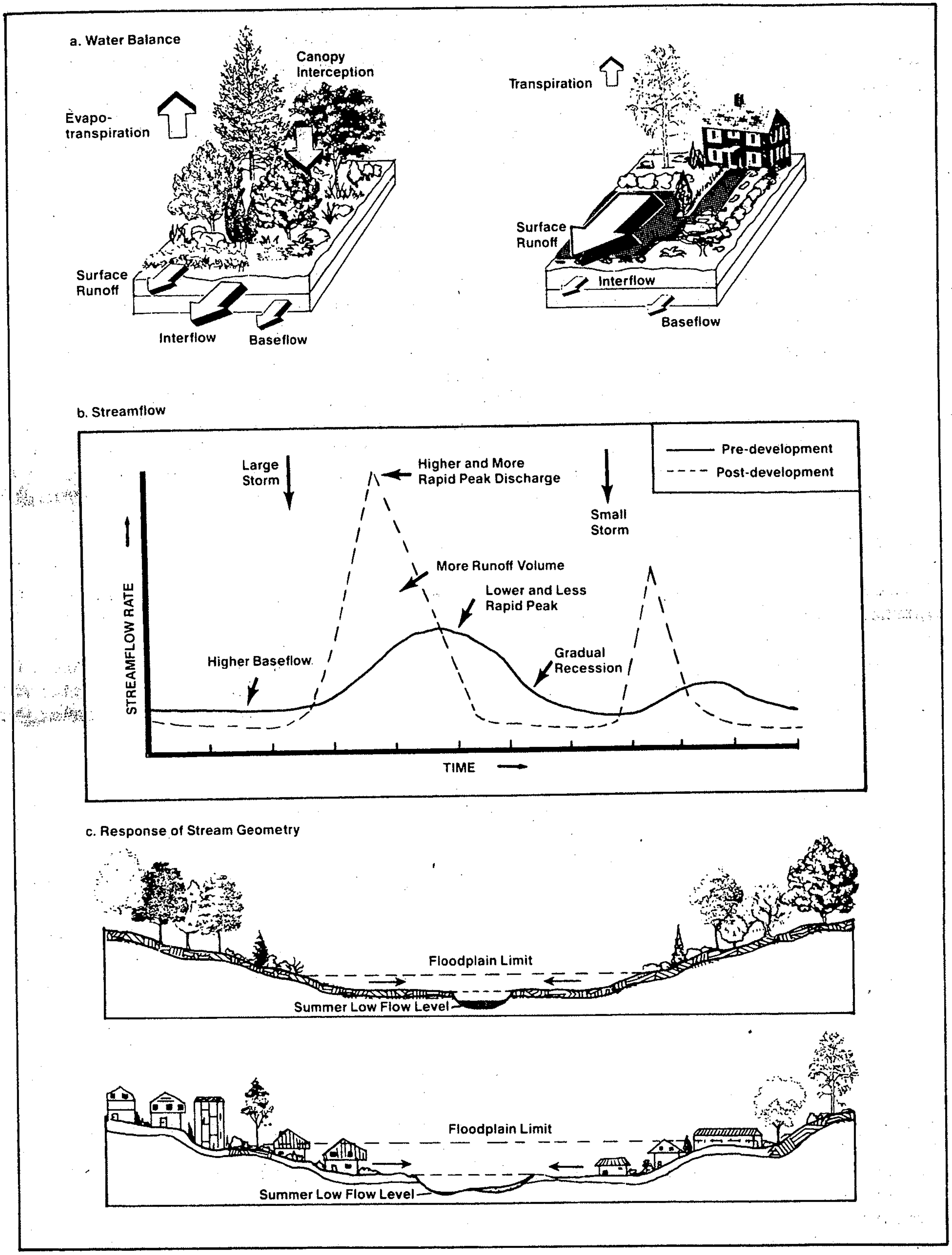
- Streambanks are gradually undercut and slump into the channel. Trees that had protected the banks are exposed at the roots, and are more likely to be windthrown, triggering a second phase of bank erosion.

- The prodigious quantities of the sediment eroded from streambanks and upland areas is seldom completely exported from the watershed. Much of it remains as temporary channel storage in the form of sandbars and other sediment deposits. Gradually, the extra sediment moves through the stream network as bedload. However, for many years the channel substrate is covered by shifting deposits of mud and coarse sand.

Degradation of Aquatic Ecosystems

The aquatic ecosystems in urban headwater streams are particularly susceptible to the impacts of urbanization. The massive shift from the natural flow and channel conditions reduce the habitat value of the stream. Dietemann (1975), Ragan and Dietemann (1976), Klein (1979) and MWCOG (1982) have all tracked trends in fish diversity and abundance over time in local urbanizing streams. Each of the studies has shown that fish communities become less diverse and are composed of more tolerant species after the surrounding watershed is developed. Sensitive fish species either disappear or occur very rarely. In most cases, the total number of fish in urbanizing streams may also decline.

Figure 1.1: Changes in Watershed Hydrology as a Result of Urbanization



Similar trends have been noted among aquatic insects which are the major food resource for fish. These species cling to rocks and rely on the passing flow of leaf litter and organic matter for sustenance. Higher post-development sediment and trace metals can interfere in their efforts to gather food. Changes in water temperature, oxygen levels, and substrate composition can further reduce the species diversity and abundance of the aquatic insect community. No single factor is responsible for the progressive degradation of urban stream ecosystems. Rather, it is probably the cumulative impacts of many individual factors such as sedimentation, scouring, increased flooding, lower summer flows, higher water temperatures, and pollution.

Pollutant Export During the Construction Phase

Pollutant export increases dramatically both during and after development. Initial clearing and grading operations during construction expose much of the surface soils. Unless adequate erosion controls are installed and maintained at the site, enormous quantities of sediment are delivered to the stream channel, along with attached soil nutrients and organic matter (Pitt, 1985). Uncontrolled construction site sediment loads have been reported to be on the order of 35 to 45 tons/acre/year (Novotny and Chesters, 1981; Wolman and Schick, 1967; Yorke and Herb, 1976, 1978). By way of comparison, sediment loads from agricultural and stabilized urban land uses are one and two orders of magnitude lower, respectively.

Pollutant Export After Site Stabilization

Once the site is stabilized, pollutants accumulate rapidly on impervious surfaces and are easily washed off. The primary source of most pollutants is from the atmosphere, in the form of wetfall and dryfall. Once deposited, up to 90% of the atmospheric pollutants deposited on impervious surfaces are delivered to receiving waters (Oberts, 1985). In most areas, the annual load of pollutants deposited from the atmosphere equals or exceeds the total load exported in runoff. Measured rates of atmospheric deposition of pollutants in the Washington area are summarized in Table 7.1.

The various surfaces of the urban landscape are also an important source of many pollutants. Trace metals, for example, are a common component of many urban surfaces, such as flashing and other roofing materials, downspouts, galvanized pipes, metal plating, paints, wood preservatives, catalytic converters, brake linings, and tires. Over time, these surfaces corrode, flake, decay, dissolve or leach out, enabling the metals to wash away in urban runoff. This process is often exacerbated by the acidity of the rainfall (pH 3.9 to 4.5 in the Washington, D.C. area; OWML, 1983; BRPC, 1986a).

Other sources of pollutants that accumulate and subsequently wash off impervious surfaces include pet droppings, vegetative matter, litter and debris. Several studies suggest that as neighborhoods become mature, some of these sources can become very important (BRPC, 1986). Litter generation and pet dropping rates increase, and the general level of "urban housekeeping" often declines as neighborhoods grow older (Syrek, 1981; BRPC, 1986a). Poor housekeeping is easier to define than to control. For example, heavy use creates bare spots that erode, dumpsters are overloaded, out of sight alleyways and service areas are not kept up, used motor oil is dumped into storm sewers, homeowners apply excessive quantities of pesticides and fertilizers, and so on. The Citizens Program for the Chesepeake Bay's <u>Baybook</u> (CPCB, 1985) is an excellent guide that shows how homeowners can improve the level of their urban housekeeping, and, consequently, improve the quality of urban runoff.

Older neighborhoods tend to become more impervious over time, as each new deck, patio, driveway, infill development and road improvement is constructed. Also, as both intended landscaping and "weed" trees grow older and become more widespread, their leaves and pollen (which would normally be slowly converted to humus on the forest floor) are more likely to fall on impervious surfaces and be washed into the channel. During the growing season, nutrients leach from tree leaves and stems during storms, and are quickly conveyed to the stream if the trees' drip line extends over an impervious area.

IMPACTS OF URBAN POLLUTANTS ON RECEIVING WATERS

The net effect of urbanization is to increase pollutant export by at least an order of magnitude over pre-development levels. The impact of the higher export is felt not only on adjacent streams, but also on downstream receiving waters such as lakes, rivers and estuaries. The nature of the impacts associated with specific urban pollutants are reviewed below. Also, the development situations that are likely to result in the most severe receiving water impacts are identified. Planners and designers should become familiar with these situations so that they can determine the pollutants of greatest concern and then choose the most appropriate BMP for the site.

Sediment

High concentrations of suspended sediment in streams cause many adverse consequences including increased turbidity, reduced light penetration, reduced prey capture for sight feeding predators, clogging of gills/filters of fish and aquatic invertebrates, reduced spawning and juvenile fish survival, and reduced angling success. Additional impacts result after sediment is deposited in slower moving receiving waters, such as smothering of the benthic community, changes in the composition of the bottom substrate, more rapid filling of small impoundments which create the need for costly dredging, and reduction in aesthetic values. Sediment is also an efficient carrier of toxicants and trace metals. Once deposited, pollutants in these enriched sediments can be remobilized under suitable environmental conditions posing a risk to benthic life (Gavin and Moore, 1982).

The greatest sediment loads are exported during the construction phase of any development site. On stabilized develpoment sites, the greatest sediment loads are exported from larger, intensively developed watersheds, that are not served by BMPs that effectively control streambank erosion.

Nutrients

Excess levels of phosphorus and nitrogen in urban runoff can lead to undesirable algal blooms in downstream receiving waters (also known as eutrophication). Generally, phosphorus is the controlling nutrient in freshwater systems. Bioassays (OWML, 1983) have indicated that the typical nutrient concentrations in urban runoff are more than sufficient to stimulate excessive algal growth. A major reason is that a majority of the nutrients in urban runoff are present in soluble forms that are readily taken up by algae.

The greatest risk of eutrophication is in urban lakes and impoundments that have long retention times (2 weeks or greater). Under optimal environmental growing conditions, these lake systems can experience chronic and severe eutrophic symptoms such as surface algal scums, water discoloration, strong odors, depressed oxygen levels (as the bloom decomposes), release of toxins, and reduced palatability to aquatic consumers. High nutrient levels also promote the growth of dense mats of green algae that attach to rocks and cobbles in shallow, unshaded headwater streams. Finally, nutrient loads from urban runoff, in combination with other sources, can contribute to eutrophication in both fresh and tidal waters. Regional examples in the Washington, D.C. area include the Occoquan and Little Seneca reservoirs, the upper Potomac and Patuxent estuaries, and the Chesapeake Bay.

As a general rule of thumb, nutrient export is greatest from development sites with the most impervious area. Exceptions include land uses that receive unusually high fertilizer inputs, such as golf courses, cemeteries, and other intensively landscaped areas.

Bacteria

Bacterial levels in undiluted urban runoff exceed public health standards for water contact recreation almost without exception. Bacteria standards violations are also a routine occurrence during storms in most of the urban streams monitored in the Washington region (MWCOG, 1984). Because bacteria multiply faster during warm weather, it is not uncommon to find a twenty-fold difference in bacterial levels between summer and winter. (US EPA, 1983). Even though bacterial levels are very high, there is some debate whether the kinds of bacteria found in urban runoff really present a severe health hazard (BRPC, 1986b). The test itself is only a count of coliform bacteria, which are an indirect and often imprecise indicator that more potent pathogens and viruses might be present (US EPA, 1983).

Although nearly every urban and suburban land use exports enough bacteria to violate health standards, older and more intensively developed urban areas produce the greatest export. The problem is especially significant in urban areas that experience combined or sanitary sewer overflows that export bacteria derived from human wastes.

Oxygen Demand

Decomposition of organic matter by microorganisms depletes dissolved oxygen (DO) levels in slower moving receiving waters such as lakes and estuaries. The degree of potential DO depletion is measured by the biochemical oxygen demand (BOD) test that expresses the amount of easily oxidized organic matter present in water. Unfortunately, the BOD test is somewhat unreliable for measuring the oxygen demand of urban runoff since trace metals may inhibit bacterial growth and thus interfere with the test (OWML, 1982). The simpler chemical oxygen demand (COD) test, which measures all the oxidizable matter present in urban runoff, is not much better, since it includes some organic matter that does not ordinarily contribute to oxygen demand, and is only weakly correlated with BOD levels (OWML, 1982).

Despite the problems in measuring oxygen demand, it is clear that urban runoff can severely depress DO levels after large storms. BOD levels can exceed 10 to 20 mg/l during storm "pulses" which can lead to anoxic conditions (zero oxygen) in shallow, slow-moving or poorly-flushed receiving waters. The problem is particularly acute in some older urban areas, where pulses of storm runoff BOD mix with overflows from combined or sanitary sewers. Chronic examples in the Washington-Baltimore region include the lower Anacostia River in the District of Columbia and the mouth of Jones Falls in Baltimore.

The greatest export of BOD occurs from older, highly impervious residential areas with outdated combined storm sewers and large populations of pets. In contrast, only moderate BOD export has been reported from newer, low density suburban residential development.

Oil and Grease

Oil and grease contain a wide array of hydrocarbon compounds, some of which are known to be toxic to aquatic life at low concentrations (Stenstrom et al., 1984). The major source of hydrocarbons in urban runoff is through leakage of crankcase oil and other lubricating agents from the automobile (Tanacredi and Stainken, 1981). As might be expected, hydrocarbon levels are highest in the runoff from parking lots, roads, and service stations. Residential land uses generate less hydrocarbon export, although illegal disposal of waste oil into storm sewers can be a local problem.

While hydrocarbons have never been routinely monitored in Washington, D.C. area storm runoff, numerous studies in other regions of the country (Hoffmann et al., 1984; Stenstrom et al., 1984, and references cited therein) have reported average hydrocarbon levels during storms ranging from 2-10 mg/l. Hydrocarbons are lighter than water and are initially found in the form of a rainbow colored film on the water's surface. However, hydrocarbons have a strong affinity for sediment, and much of the hydrocarbon load eventually adsorbs to particles and settles out. If not trapped by BMPs hydrocarbons tend to rapidly accumulate in the bottom sediments of lakes and estuaries (Wakeham, 1977; Tanacredi and Stainken, 1981), where they may persist for long periods of time, and exert adverse impacts on benthic organisms (Whipple and Hunter, 1979).

The precise impacts of hydrocarbons on the aquatic environment are not well understood. Remarkably few toxicity tests have been performed to examine the effect of urban runoff hydrocarbon loads on aquatic communities under the typical exposure conditions found in urban streams. Bioassay data which does exist is largely confined to laboratory exposure tests for specific hydrocarbon compounds, which are difficult and expensive to routinely measure in the field. Clearly, community level toxicity testing for hydrocarbons should be a high research priority, both in the water column and sediment layer.

Trace Metals

Trace metals are primarily a concern because of their toxic effects on aquatic life, and their potential to contaminate drinking water supplies. As noted before, most of the metals found in urban runoff are derived from "leakage" of the urban landscape.

A wide variety of trace metals were found in urban runoff samples taken during the special trace metals sampling program conducted as part of the Washington, D.C. area and national Nationwide Urban Runoff Program (NURP) studies. Specifically, the following metals were measured in detectable concentrations: arsenic, beryllium, cadmium, chromium, copper, cyanide, mercury, nickel, lead, selenium, thallium, and zinc (JTC, 1982; DDN, 1982). With the significant exceptions of lead, cadmium, copper and zinc, most of the trace metals were found in only a few samples, and then only in minute amounts that were well below human health or aquatic life criteria. Lead, copper and zinc were generally found in most samples, and were occasionally recorded at levels an order of magnitude higher than recommended aquatic life criteria. Maximum reported trace metal concentrations in the Occoquan Watershed Monitoring Laboratory (OWML) study of Washington area urban runoff concentrations were 0.720 mg/l for lead, 1.20 mg/l for zinc, 0.310 mg/l for copper and 0.117 mg/l for cadmium (OWML, 1983).

A few caveats should be kept in mind when evaluating the risks of trace metals in urban runoff. First, a large fraction (often over half) of the trace metals are attached to sediment. This effectively reduces the level which is immediately available for biological uptake and subsequent bioaccumulation. Metals associated with the sediment rapidly settle out of the water column and accumulate in soils and aquatic sediments (OWML, 1983; Gavin and Moore, 1982). Second, urban runoff events typically occur over a shorter duration (2 to 8 hours) than the exposure intervals used in aquatic bioassay tests (24 hours to a week for chronic toxicity criteria). Third, urban runoff is often subject to substantial dilution after mixing with other runoff sources. Nonetheless, it is likely that trace metals are toxic to stream life in certain situations (JTC, 1982), particularly for the more soluble metals such as copper and zinc.

An interesting note concerning metals concentrations is that both atmospheric and storm runoff levels of lead in the Washington region have dropped by about 50% since the late 1970s (MWCOG, 1983b). The sharp drop is believed to be due to the increased use of unleaded gasoline in the early 1980s, that has historically been the major source of lead to the environment.

Toxic Chemicals

A "priority pollutant scan" was conducted during both the national and Washington area NURP studies to determine the presence of over 120 toxic or carcinogenic chemicals and compounds. While very limited in scope, the scans rarely turned up toxic chemicals in amounts that exceeded current safety criteria (DDN, 1982; JTC, 1982). The urban runoff scans were primarily conducted in suburban residential areas not expected to have many sources of toxic pollutants (with the possible exception of illegally disposed or applied household hazardous wastes, such as waste oil, paint thinners, preservatives and pesticides). The Washington area scan indicated the presence of ten pesticides in urban runoff, but the concentrations were near the limits of detection (less than 1 ppb) (OWML, 1983). Other priority pollutants detected in Washington area runoff samples included bis(2-ethylhexyl) phthalate (a widely used plasticizer that leaches from plastic products), and several phenols and cresols (associated with wood preservatives). All of the priority pollutants detected in the Washington, D.C. scan (with the exception of trace metals) were well below relevant criteria or guidelines.

While the priority pollutant scans generally indicated that exotic chemicals are not commonly found in residential runoff, it should be remembered that the limited number of samples analyzed were not collected from existing or abandoned industrial areas that might serve as a greater source of toxicants.

Chlorides

Chlorides or salts are often introduced into streams after they are applied to remove ice and snow from roads, parking lots and sidewalks. Salt levels in snowmelt runoff have been reported to exceed several thousand milligrams per liter (about as salty as the Chesapeake Bay) (Novotny and Jones, 1986; Scott, 1981). Due to its extreme solubility, almost all the chloride applied for snow removal purposes ends up in surface or ground waters (Pitt, 1985). At high levels, chlorides are toxic to many freshwater aquatic organisms, as they are only adapted to withstand a relatively narrow range of salinity.

Thermal Impacts

Elevated water temperatures can have dire consequences for stream biota which are adapted to a coldwater environment. A rise in water temperature of just a few degrees Celsius over ambient conditions can reduce or eliminate sensitive stream insects and fish species, such as stoneflies, mayflies and trout. In general, sustained summertime water temperatures in excess of 21 degrees Celsius (70 degrees Fahrenheit) are considered to be stressful, if not lethal, to many coldwater organisms. Thermal enrichment problems are critical for many Piedmont streams that straddle the geographic and/or thermal borderline between coldwater and warmwater stream conditions.

A number of factors can increase summertime water temperatures in urban headwater streams. Of these, three factors often act synergistically to increase water temperatures. First, as the urban landscape heats up on warm summer days, it tends to impart a great deal of heat to any runoff passing over it. Second, fewer trees are present on the streambank to shade the stream channel, adding to the warming effect. Third, runoff stored in shallow wet ponds and other impoundments is heated in between storms, and then may be released in a rapid pulse, following a storm.

ESTIMATING URBAN STORMWATER POLLUTANT EXPORT

This section presents a Simple Method for estimating pollutant export from urban development sites. The Simple Method is empirical in nature, and utilizes the extensive database obtained in the Washington, D.C. area NURP study, as well as the national NURP data analysis (MWCOG, 1983b; US EPA, 1983; Driscoll, 1983a). The Simple Method is versatile in that it predicts pollutant loadings under a variety of planning conditions. It can also be used to estimate the probability that pollutant concentrations exceed a given threshold level.

The Simple Method is primarily intended for use on development sites less than a square mile in area. Moreover, the Simple Method is designed to provide an quick, easy and versatile means for estimating pollutant loads. Therefore, the method sacrifices some precision for the sake of simplicity and generality. Despite its limitations, the Simple Method is considered precise enough to make reasonable and reliable nonpoint pollution management decisions at the site-planning level. Examples of how to use the method are provided at the end of the section. Additional documentation on the derivation of the Simple Method is presented in detail in Appendix A.

Storm pollutant export (L, in pounds) from a development site can be determined by solving the following equation:

(EQ 1.1) $$L = [(P)(Pj)(Rv)/12](C)(A)(2.72)$$

where P = rainfall depth (inches) over the desired time interval.
Pj = factor that corrects P for storms that produce no runoff.
Rv = runoff coefficient, which expresses the fraction of rainfall which is converted into runoff.
C = flow-weighted mean concentration of the pollutant in urban runoff (mg/l).
A = area of the development site (acres).

12, 2.72 are unit conversion factors.

The user need only define five parameters, each of which are readily determined from site plan data, or are constants:

P (depth of rainfall)

The value of P selected depends on the time interval over which loading estimates are desired. For a normal year of rainfall, P will be about 40 inches in the Washington, D.C. area. Values of 30 and 50 inches can be used to characterize extremely dry and wet years, respectively. Long-term rainfall records from National Weather Service (NWS) stations should be used to estimate P in other regions of the country. If a load estimate is desired for a specific design storm or year of record, then the user can supply the relevant value of P.

Pj (correction factor)

The value of Pj is used to account for the fraction of annual or seasonal rainfall that does not produce any measurable runoff. Approximately 50% of the storms each year drop less than two-tenths of an inch of precipitation. Storms of this size are often not sufficient to create runoff; the rainfall is stored in surface depressions that eventually evaporate. An analysis of Washington, D.C. area rainfall/runoff patterns (Appendix A, Section 7) suggests that only 90% of rainfall events produce any runoff. Therefore, Pj should be set to 0.9 for annual and seasonal calculations. For individual storms, Pj should be set to 1.0 to avoid double counting.

Rv (runoff coefficient)

Rv is the measure of site response to rainfall events, and is calculated as:

(EQ 1.2) $Rv = r/p$

where r = storm runoff (inches).
p = storm rainfall (inches).

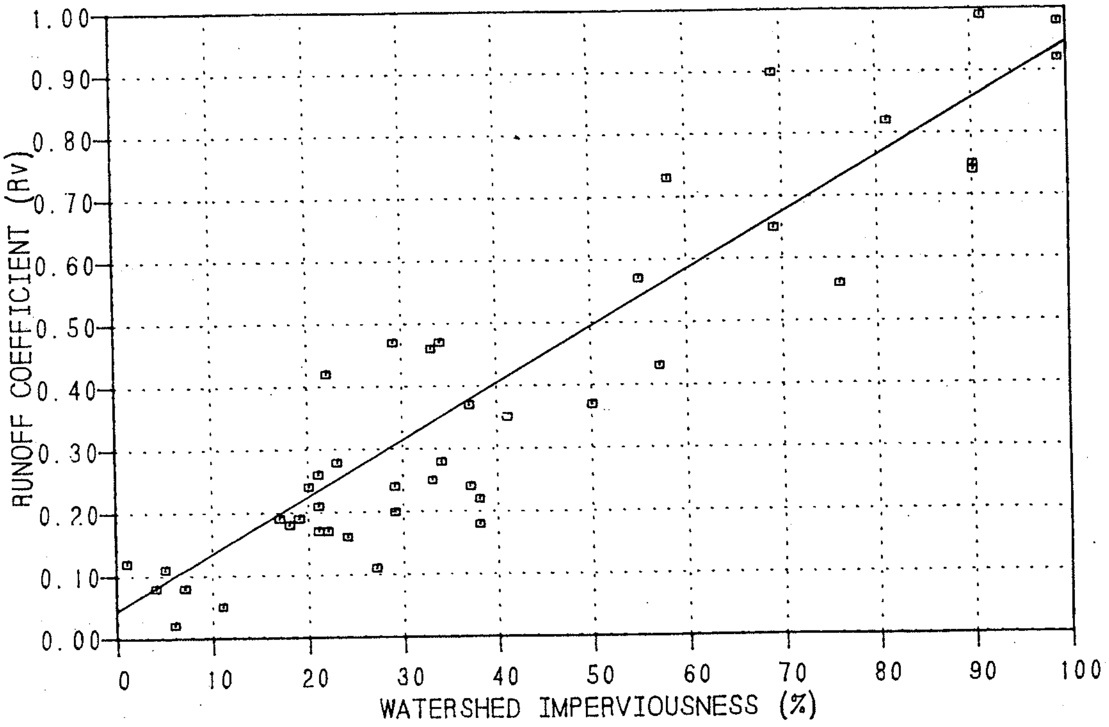
The Rv for a site depends on the nature of the soils, topography, and cover. However, the primary influence on the Rv is the degree of watershed imperviousness. Figure 1.2 shows the relation between the mean Rv and the degree of watershed imperviousness for 47 small urban catchments monitored throughout the region and the nation (Appendix A, Section 6). Although some scatter is evident in the plot, watershed imperviousness (I) does appear to be a reasonable predictor of the Rv. The following equation represents the best fit line through the dataset (adjusted R^2=0.71):

(EQ 1.3) $Rv = 0.05 + 0.009(I)$

where I = the percent of site imperviousness.

Values for I are readily obtained from site plans or accompanying hydrological computations. This is done by summing the area of the site covered by structures, sidewalks, driveways, parking lots, roads, patios and other impermeable areas (by planimetry or square counting) and dividing it by the total site area.

Figure 1.2: Relationship Between Watershed Imperviousness (I) and the Storm Runoff Coefficient (Rv)



NOTE: 44 small urban catchments monitored during the national NURP study. (For mean values, see Table A.5).

A (site area)

The total area of the site (in acres) can be obtained from site plans. Caution should be exercised if site area (A) is greater than one square mile (640 acres). Alternate procedures, outlined in Section 6 of Appendix A, should be employed in these larger watersheds to account for baseflow runoff volumes.

C (pollutant concentration)

Statistical analysis of over 300 runoff events monitored during the 1980-81 Washington, D.C. area NURP project indicated that there was; 1) no significant difference in average pollutant concentrations between the eight widely different urban sites measured, and 2) no consistent correlation between pollutant concentrations and storm volume or intensity (Appendix A, Section 5). A similar analysis conducted on the much larger national NURP database reached the same conclusions (US EPA, 1983; Driscoll, 1983a). The practical implication of these findings is that a single concentration value (C) can be applied for purposes of estimating pollutant loads.

Average, flow-weighted C values for selected pollutants measured during the NURP study are presented in Table 1.1. These values generally represent the pollutant levels emanating from stabilized, relatively recent suburban development sites in the region. These values, however, may not always be appropriate for all site conditions. Alternative values of C for special watershed conditions are presented in Table 1.1 and are described below.

1. OLDER, POORLY MAINTAINED URBAN NEIGHBORHOODS often have significantly higher pollutant concentrations during storms than newer developments. Mean pollutant levels monitored in five older residential catchments in downtown Baltimore, Maryland (BRPC, 1986b) are provided in Table 1.1. As can be seen, the pollutant levels in downtown Baltimore are approximately 2 to 5 times higher than those reported for suburban Washington, D.C., and are attributed to poor urban "housekeeping" (i.e., poor trash removal, accumulation of debris, deteriorating housing stock, high traffic volumes, poor upkeep of lawns and open space). These C values can be used in lieu of the suburban Washington C values when assessing loads for older urban residential areas.

2. HIGHLY IMPERVIOUS CENTRAL BUSINESS DISTRICTS generate urban pollutant levels that are also slightly higher than the suburban Washington C values. This is due in part to greater traffic volume and higher atmospheric loading rates (MWCOG, 1983a). Pollutant levels monitored in the K Street corridor of Washington, D.C. are provided in Table 1.1. Nutrient, BOD and trace metal levels measured during 27 storms at two sites were often higher than other residential or commercial suburban sites monitored in the Washington, D.C. NURP study (MWCOG, 1983b; NVPDC, 1981).

3. FOREST C values are provided for comparative purposes in Table 1.1, and were obtained from extensive storm monitoring of pollutant levels in several small forested watersheds in the Occoquan basin in Northern Virginia (OWML, 1983). These C values can be used to roughly estimate "natural" background storm loadings contributed from undeveloped areas to aid in assessing how much pollutant export will increase as a result of urban development activity.

4. NATIONAL RUNOFF CONCENTRATIONS were obtained from over 2300 storms monitored at 22 NURP project sites across the nation (US EPA, 1983). These average values are recommended for use in areas outside of the Middle Atlantic states. The national C values are slightly higher than the values for new suburban sites in the Washington, D.C. area, and slightly lower than values for the older urban areas of Baltimore.

5. NATIONAL URBAN HIGHWAY CONCENTRATIONS were computed from over 250 storm EMC samples collected at eight urban highway sites across the nation as part of a Federal Highway Administration study (Shelley and Gaboury, 1986). The high concentration of metals and phosphate apparently reflects the impact of vehicle emissions. The same study indicated that pollutant concentrations in rural highway runoff were typically one half of the NURP urban runoff average.

The Simple Method has been designed such that any urban storm monitoring dataset can be used as a basis for estimating loads. Thus, if newer or more site specific pollutant concentration data becomes available in the future, the Simple Method can be easily modified to incorporate the new C values. The appropriate procedures for developing new C values are given in Section 3 of Appendix A.

Table 1.1: Urban 'C' Values For Use With the Simple Method (mg/l)

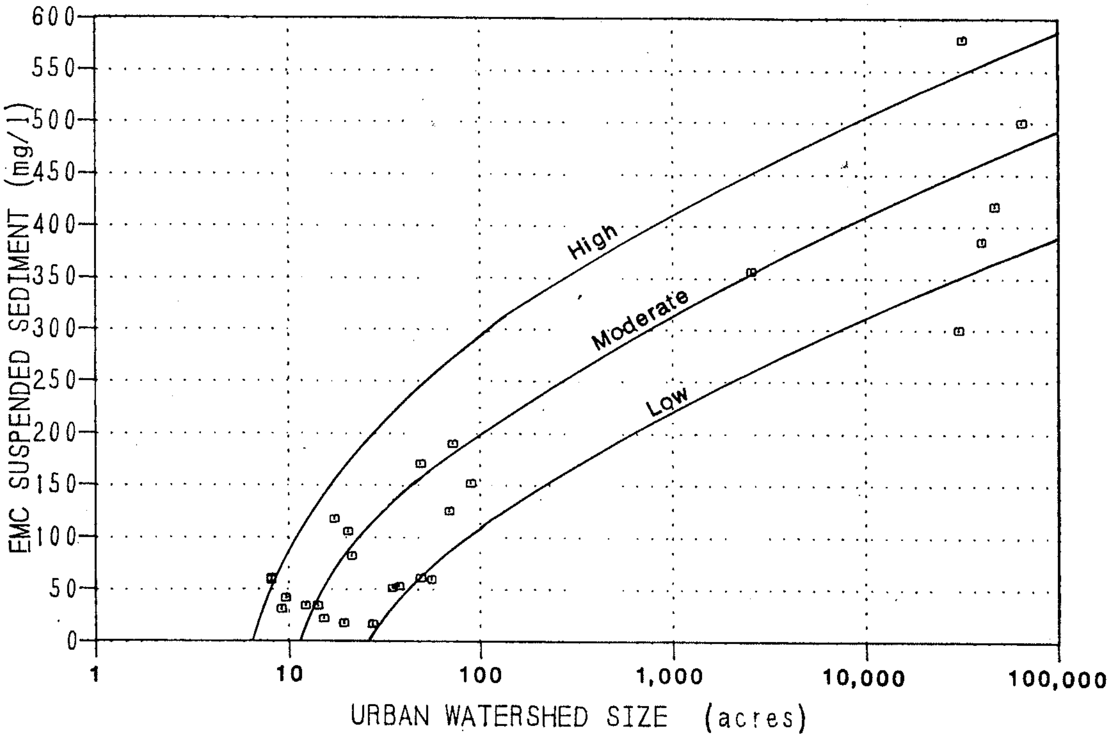
POLLUTANT	NEW SUBURBAN NURP SITES (Wash.,DC)	OLDER URBAN AREAS (Baltimore)	CENTRAL BUSINESS DISTRICT (Wash.,DC)	NATIONAL NURP STUDY AVERAGE	HARDWOOD FOREST (Northern Virginia)	NATIONAL URBAN HIGHWAY RUNOFF
PHOSPHORUS						
Total	0.26	1.08	-	0.46	0.15	-
Ortho	0.12	0.26	1.01	-	0.02	-
Soluble	0.16	-	-	0.16	0.04	0.59
Organic	0.10	0.82	-	0.13	0.11	-
NITROGEN						
Total	2.00	13.6	2.17	3.31	0.78	-
Nitrate	0.48	8.9	0.84	0.96	0.17	-
Ammonia	0.26	1.1	-	-	0.07	-
Organic	1.25	-	-	-	0.54	-
TKN	1.51	7.2	1.49	2.35	0.61	2.72
COD	35.6	163.0	-	90.8	>40.0	124.0
BOD (5-day)	5.1	-	36.0	11.9	-	-
METALS						
Zinc	0.037	0.397	0.250	0.176	-	0.380
Lead	0.018	0.389	0.370	0.180	-	0.550
Copper	-	0.105	-	0.047	-	-

Predicting Suspended Sediment Levels

Suspended sediment concentrations in urban runoff cannot be predicted by the previously described methods. The only predictive relationship to emerge from the Washington, D.C. NURP study was that sediment levels are generally related to watershed size. As shown in Figure 1.3, mean storm sediment concentrations tend to increase with drainage area in 25 urban watersheds in the region (watershed areas ranging from 5 to 100,000 acres in size) (Hickman, 1984; OWML, 1983; MWCOG, 1983b; NVPDC, 1979).

Higher storm sediment levels in larger watersheds are primarily due to bank and channel erosion, rather than erosion of pervious areas by overland flow or washoff of sediments from impervious areas within the watershed. Under this theory, as watershed size becomes larger, the length of the stream channel network and the susceptibility to channel erosion increases markedly. Most small headwater streams in the region have abundant supplies of stored sediment that have been gradually deposited by previous centuries of agricultural erosion, or more recently, by construction-related erosion (Meade, 1982; Costa, 1975; Wolman and Schick, 1967). The large quantities of sediment in channel storage can become resuspended and transported out of the watershed by the increased severity of flooding that often follows urbanization.

Figure 1.3: Relationship Between Watershed Area and Sediment Event Mean Concentration (EMC)



Alternative sources of sediment (pervious area erosion and/or impervious area washoff) do not appear to be capable of producing high sediment concentrations. Erosion of pervious areas in most stabilized urban areas is minimized by the extensive cover of lawns and open space, and washoff from impervious areas is limited by the atmospheric supply of solids, which amounts to less than a tenth of a ton per year (see Table 7.1). Both sources are probably responsible for the relatively low suspended sediment concentrations (15-25 mg/l) reported for very small watersheds.

The relationship between mean storm sediment levels and drainage area provides a first-cut estimate of the expected storm sediment concentrations for specific development situations. The rather wide band drawn around the data points in Figure 1.3 reflects the wide variability observed in the field. The choice of a high, moderate or low value on the curve is a matter of subjective interpretation, but some general guidance is offered in Table 1.2 below.

Table 1.2: Watershed Channel Network Condition

CRITERIA	LOW EMC	MODERATE EMC	HIGH EMC
STABILITY CONDITION OF CHANNEL	vegetated swales or storm sewers	intermediate	open channel, cut banks alternating w/ channel sandbars, fallen trees
CHANNEL SEDIMENT STORAGE	small deposits in storm drains, stabilized land use	"	large silt or clay deposits, evidence of recent or ongoing construction, water becomes murky after disturbing bottom
STREAM VELOCITY	low slope, low imperviousness	"	high slope, high watershed imperviousness

ESTIMATING URBAN POLLUTANT THRESHOLDS AND FREQUENCY

In some cases, the concentration of an urban pollutant, rather than the annual load, is needed to assess a water quality problem. This is often the case with trace metals and toxicants which primarily impact the environment when their ambient concentration exceeds a critical threshold over some specified period of time. It is more useful then, to express the delivery of these urban pollutants in terms of an expected maximum concentration which recurs over a given time interval. For example, trace metal X, exceeds a critical concentration threshold Y, in Z percent of all storms.

Data of this nature was produced using statistical procedures outlined in US EPA (1983) and Section 2 of Appendix A in this manual, for the entire Washington, D.C. NURP database of over 300 runoff events. The results are detailed for selected pollutants in Table 1.3. Note that the exceedance frequency refers to the percent of runoff events in which a given concentration level is equaled or exceeded. Since, on average, there are about 65 measurable runoff events per year in the Washington, D.C. area (Appendix A, Section 7), the exceedance frequency can be converted into a return interval by the following equations:

(EQ 1.4) $e = (p)(St)$ and,

(EQ 1.5) $r = 1/[(p)(St)] = 1/e$

where
e = number of exceedances per year.
p = number of exceedance events/number of runoff events.
St = number of runoff events/year (average of 65).
r = recurrence interval (years) for a given exceedance.

The pollutant levels shown in Table 1.3 are rarely experienced in downstream receiving waters due to dilution and adsorption. Mancini (1983) and US EPA (1983) provide useful screening procedures for treating these factors, and an example calculation is provided later in this chapter.

Table 1.3: Exceedance Frequency (e) for Selected Urban Pollutant

POLLUTANT CONCENTRATION (mg/l)	PERCENT OF STORMS IN WHICH GIVEN CONCENTRATION IS EXCEEDED				
	50%	25%	10%	5%	1%
SEDIMENT	31	71	151	235	545
TOTAL PHOSPHORUS	0.27	0.43	0.65	0.82	1.31
TOTAL NITROGEN	2.2	3.2	4.5	5.6	8.2
COD	42	61	84	103	149
LEAD	0.021	0.042	0.076	0.109	0.149
COPPER	0.010	0.020	0.037	0.055	0.114
ZINC	0.06	0.101	0.161	0.216	0.355

For reference purposes, calculated threshold concentrations can be compared to existing water quality standards and criteria, some of which are shown in Table 1.4. These criteria were developed by US EPA (1983) to attempt to define the effects of short duration and intermittent exposures typical of urban runoff. As one might guess from the large spread in the numbers, there is considerable scientific uncertainty about the chronic or immediate effects of trace metals on aquatic life.

Table 1.4: U.S. EPA Trace Metal Criteria For Urban Runoff Exposure

TRACE METAL CONTAMINANT	WATER[1] HARDNESS (mg/l as CACO3)	HUMAN[2] INGESTION (Food/Drink) (ug/l)	AMBIENT LIFE CRITERIA FOR INTERMITTENT EXPOSURE (ug/l)[3] Threshold[4] Effect	Significant[5] Mortality
Copper	50	-	20	50-90
	100	-	35	90-150
	200	-	80	120-350
Cadmium	50	10	3	7-160
	100	10	6.6	15-350
	300	10	20	45-1070
Lead	50	50	150	350-3200
	100	50	360	820-7500
	200	50	850	1950-17850
Zinc	50	-	380	870-3200
	100	-	680	1550-4500
	200	-	1200	2750-8000
Nickel	-	13.4	-	-

[1] See glossary for definition.

[2] Derived from EPA drinking water criteria.

[3] EPA estimate of toxicity under intermittent, short duration exposure (several hours once every several days).

[4] Concentration causing mortality to the most sensitive individual of the most sensitive species.

[5] Significant mortality shown as a range: 50% mortality in the most sensitive species, and mortality of the most sensitive individual in the species in the 25th percentile of sensitivity.

EXAMPLE 1-1: ESTIMATING THE INCREASED NUTRIENT LOADING FROM AN UNCONTROLLED DEVELOPMENT SITE

Given a 38 acre development site being converted from woodland to a townhouse community (total imperviousness-45%), what will the annual post-development nitrogen and phosphorus loads be during a normal year of rainfall? How great of an increase is this over the pre-development nutrient loads?

Step 1. Estimate Parameters For Pre- and Post-Development Conditions. Use Washington, D.C. suburban and forest C values (from Table 1.1). Compute Rv using general equation (EQ 1.3). (Assume I= 2% for forest).

Parameter	Pre-Development	Post-Development
P	40 inches	40 inches
Pj	0.9	0.9
Rv	0.05 + .009(2)= 0.07	0.05 + .009(45)= 0.46
C (total N)	0.78 mg/l	2.00 mg/l
C (total P)	0.15 mg/l	0.26 mg/l
A	38 acres	38 acres

Step 2. Compute Annual Storm Loads (EQ 1.1).

$$L = [(P)(Pj)(Rv)/12](C)(A)(2.72)$$

Pre-Development:

$$TN = [(40)(0.9)(0.07)/12](0.78)(38)(2.72) = 17 \text{ pounds/year}$$

$$TP = [(40)(0.9)(0.07)/12](0.15)(38)(2.72) = 3 \text{ pounds/year}$$

Post-Development:

$$TN = [(40)(0.9)(0.46)/12](2.0)(38)(2.72) = 285 \text{ pounds/year}$$

$$TP = [(40)(0.9)(0.46)/12](0.26)(38)(2.72) = 37 \text{ pounds/year}$$

Step 3. Calculate Increase in Storm Nutrient Loads After Development.

As a result of the development of the site, annual storm nutrient loadings are expected to increase by 268 and 34 pounds/year for nitrogen and phosphorus, respectively. In reality, the increase in storm nutrient loads may be slightly less since the pre-development land use (forest) often exports a modest nutrient load in baseflow. Even so, the large change in nutrient loads as a result of the development could very likely cause eutrophication or nutrient enrichment of downstream receiving waters, particularly if they are slow-moving or poorly-flushed. A more precise analysis of receiving water response to the increased nutrient load would be warranted. The simple nutrient loading response model described by Reckhow (1980) or other similar models could be used for this purpose.

EXAMPLE 1-2: DETERMINING LONG-TERM SEDIMENT ACCUMULATION IN A WET POND

A planner wants to know how much storage volume will be eventually lost due to sediment deposition in a 7,500 cubic yard wet pond draining a 106 acre, 55% impervious watershed over a twenty year period. Assume that 1) the average sediment removal efficiency of the pond is 60%, 2) one ton of sediment eroded from the watershed fills a volume equivalent to one cubic yard within the pond, 3) the open channel network within the watershed is in poor condition, 4) the average annual rainfall is 40 inches.

SOLUTION: From Figure 2, the expected mean sediment concentration for a 100 acre watershed in poor condition is about 280 mg/l. The post-development storm runoff coefficient (Rv) will be 0.55. Therefore, the annual sediment load during a normal year of rainfall can be obtained by solving the general equation (EQ 1.1).

$$L = [(P)(Pj)(Rv)/12](C)(A)(2.72)$$

$$L = [(40)(0.9)(.55)/12](280)(106)(2.72) = 133{,}200 \text{ pounds} \quad (67 \text{ tons/year})$$

If the pond is 60% efficient in trapping sediment, the total load delivered over twenty years would be:

(67 tons/yr)(20 yr)(0.6) = 800 tons.

The trapped sediment load would fill up about 800 cubic yards, or about 11% of the ponds's total stormwater storage capacity.

It should be noted that the sediment load calculated represents the sediment load from a stabilized urban watershed and its channel network. It does not include any sediment supplied during the construction phase, which can be extremely large load. Construction phase sediment loads can be roughly calculated assuming a sediment C of 10,000 mg/l (Yorke and Herb; 1976), and an Rv of 0.9 per exposed acre. Thus, the annual uncontrolled sediment load during construction is on the order of:

$$L = [(40)(0.9)(0.9)/12](10{,}000)(106)(2.72)/2000 = 3890 \text{ tons/year} \quad (37 \text{ tons/acre/yr}).$$

This estimate falls within the field measurements of average sediment loss from uncontrolled construction sites reported by Wolman and Schick (1967) and Yorke and Herb (1976) for the Washington, D.C. area of 30 to 40 tons per acre. The calculation dramatically illustrates the need for effective on-site sediment controls during construction.

EXAMPLE 1-3: EVALUATING THE POSSIBLE EXCEEDANCE OF TRACE METAL WATER QUALITY CRITERIA IN A SENSITIVE TROUT STREAM

A planner wants to evaluate whether a proposed development will increase levels of trace metals sufficiently to adversely impact aquatic life in a trout stream. The 75 acre, 27% impervious site is situated within a 320 acre woodland watershed. The following planning conditions are assumed, 1) a storm with one inch precipitation, 2) a softwater stream with a hardness of 100 mg/l (CaCO3) 3) the effective imperviousness of the forest area is 2%, and 4) the background concentration of trace metals in the forest runoff is negligible. Determine the expected level of the trace metals lead, zinc and copper which are exceeded 5% of the time, and compare these with established instantaneous water quality criteria.

SOLUTION: Separately calculate the volume of storm runoff from the developed and forested portions of the watershed for a one-inch storm, using a modified form of the general equation (EQ 1.1):

Runoff volume (acre-feet) = (P)(Rv)(A)/12 (Note: Pj is not used for a single storm.)

DEVELOPMENT SITE: (1)(.29)(75)/12 = 1.81 ac-ft

FORESTED AREA: (1)(.07)(245)/12 = 1.43 ac-ft

The total runoff volume for the watershed would be equal to 3.24 ac-ft, and the dilution ratio (developed site runoff/watershed runoff) is 0.56.

The next step involves selecting the trace metal concentration level which is exceeded in 5% of all storms (Table 1.3), or about 3 times per year. Trace metal criteria established for protection of aquatic life are given in Table 1.4. The estimated in-stream metals concentrations can be compared to the threshold values in Table 1.4. as a first cut assessment of potential metal toxicity. As shown below, it is evident that lead and zinc levels appear "safe", while copper levels approach the criteria, even after dilution takes place.

TRACE METAL	Undiluted 5% Exceed. Conc. (C) (Table 1.3)	Stream Concentration After Correcting for Stream Dilution = (C)(0.56)	US EPA "Threshold" Criteria (Table 1.4)
Lead	109 ug/l	61 ug/l	360 ug/l
Zinc	216 ug/l	121 ug/l	680 ug/l
Copper	55 ug/l	31 ug/l	35 ug/l

Annual Storm Export of Pollutants from Development Sites

For the convenience of the reader, the Simple Method has been solved to provide estimates of urban storm pollutant export for incremental values of impervious cover. The results are shown in Table 1.5 for several urban pollutants. Table 1.5 also provides a general range of impervious cover associated with common development situations. Unless noted otherwise, all calculations assume a normal year of rainfall (P=40 inches) and suburban NURP pollutant concentration values.

Proper Use of the Simple Method

The Simple Method should provide reasonable estimates of changes in pollutant export resulting from development activity. However, several caveats should be kept in mind when applying this method:

- The Simple Method only estimates pollutant loads generated during storms. It does not consider baseflow runoff and associated pollutant loads. Typically, baseflow is negligible or non-existent at the scale of a small development site, and can be safely neglected. However, larger residential watersheds often do generate appreciable volumes of baseflow. Pollutant levels in baseflow are generally low and can seldom be distinguished from natural background levels (NVPDC, 1979). Consequently, baseflow pollutant loads normally constitute only a small fraction of the total load delivered from a site. Nevertheless, it is important to remember that load estimates derived by the Simple Method refer only to stormflow loads and should not be confused with the total load from the site, particularly when the density of watershed development is low. For example, in large low density residential watersheds (I less than 5%), as much as 75% of the annual runoff volume may occur as baseflow. In such a case, the annual baseflow nutrient load may be equivalent to the annual stormflow nutrient load (Appendix A, Section 3). Procedures for calculating baseflow runoff volumes and associated pollutant loads can be found in Appendix A, Section 6.

- The Simple Method provides estimates of storm pollutant export that are probably close to the "true" but unknown value for the site. However, it is very important not to overemphasize the precision of the results obtained. For example, it would be inappropriate to use the method to evaluate relatively similar development proposals (e.g., 34.3% I versus 36.9% I).

- The Simple Method is based on urban runoff monitoring data from recently stabilized suburban watersheds. Although modifications have been made to extend the method to older urban areas, central business districts and some natural reference areas, there are still several areas in which a reliable C value may not exist. These include construction areas, industrial areas, rural development and agricultural uses.

- The Simple Method provides a general planning estimate of likely storm pollutant export from development sites. More sophisticated methods, such as watershed and receiving water simulation modeling, may be needed to analyze larger and more complex watersheds.

Table 1.5: Annual Storm Pollutant Export For Selected Values of Impervious Cover (I) Developed from the Simple Method[1]

LAND[2] USE	SITE IMPERVIOUSNESS	TOTAL PHOSPHORUS[3]	TOTAL NITROGEN	BOD 5-day	EXTRACTABLE	
					ZINC	LEAD
		------------ pounds/acre/year ------------				
RURAL RESIDENTIAL	0	0.11	0.8	2.1	0.02	0.01
	5	0.20	1.6	4.0	0.03	0.01
	10	0.30	2.3	5.8	0.04	0.02
LARGE LOT SINGLE FAMILY	10	0.30	2.3	5.8	0.04	0.02
	15	0.39	3.0	7.7	0.06	0.03
	20	0.49	3.8	9.6	0.07	0.04
MEDIUM DENSITY SINGLE FAMILY	20	0.49	3.8	9.6	0.07	0.04
	25	0.58	4.5	11.4	0.08	0.05
	30	0.68	5.2	13.3	0.10	0.05
	35	0.77	6.0	15.2	0.11	0.06
TOWNHOUSE	35	0.77	6.0	15.2	0.11	0.06
	40	0.87	6.7	17.1	0.12	0.07
	45	0.97	7.4	18.9	0.14	0.07
	50	1.06	8.2	20.8	0.15	0.08
GARDEN APARTMENT	50	1.06	8.2	20.8	0.15	0.08
	55	1.16	8.4	22.7	0.16	0.09
	60	1.25	9.6	24.6	0.18	0.09
HIGH RISE, LIGHT COMMERICAL/ INDUSTRIAL	60	1.25	9.6	24.6	0.18	0.09
	65	1.35	10.4	26.4	0.19	0.10
	70	1.44	11.1	28.3	0.21	0.10
	75	1.54	11.8	30.2	0.22	0.11
	80	1.63	12.6	32.0	0.23	0.11
HEAVY COMMERCIAL, SHOPPING CENTER	80	1.63	12.6	32.0	0.23	0.11
	85	1.73	13.3	33.9	0.25	0.12
	90	1.82	14.0	35.8	0.26	0.13
	95	1.92	14.8	37.7	0.27	0.13
	100	2.00	15.4	39.2	0.28	0.14

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[1] P=40 inches, Pj=0.9, Rv=0.05+0.009(I), C=suburban values, A=1 acre.

[2] Rural Residential: 0.25-0.50 Dwelling Units (DU)/acre
Large Lot Single Family: 1.0-1.5 DUs/acre
Medium Density Single Family: 2-10 DUs/acre
Townhouse and Garden Apartment: 10-20 DUs/acre

[3] These values are for NEW DEVELOPMENT SITES ONLY. For older urban areas, central business districts, sites with highways, or areas outside of the Middle Atlantic region, use a more appropriate "C" value in Equation 1.1 (see Table 1.1).

CHAPTER 2: CHOOSING THE BEST BMP OPTION FOR A SITE

This chapter outlines factors that planners and engineers need to consider when choosing an urban best management practice (BMP) for a particular development site. It begins with a brief discussion of the minimum objectives that a BMP plan for a site should meet. The next section provides a series of screening tools that can be used to select the most appropriate BMP for a particular development site. These screening tools can be used to evaluate the following:

- BMP options that are suitable for a site, given its physical condition and development status.
- Stormwater control benefits provided by each BMP option.
- The expected pollutant removal capability for each BMP option, under several different design scenarios.
- Environmental and human amenity values associated with the BMP option selected.

The screening tools can be used in any order, or may be used as an overall summary of BMP performance. Several examples of how the screening tools can be used for a particular development site are presented. The chapter concludes with a summary index that shows where more information on the design, cost and maintenance of a BMP option can be found elsewhere in the manual.

OBJECTIVES IN BMP PLANNING

Over the past two decades, a number of urban BMPs have been developed and refined to mitigate some of the adverse impacts associated with development activity. Experience has shown that each BMP option has both unique capabilities and persistent limitations. These, in turn, must be balanced with both the physical constraints imposed by the development site and the overall management objectives for the watershed. In practice, this balance is achieved through a negotiating process between the engineering consultant and the local planner. Typically, the engineering consultant is responsible for developing the initial BMP plan, and represents the interests of the developer. The planner reviews the plan to ensure that it conforms with local policies and design standards, and represents the interests of the community.

During the BMP review process, it is important to identify the ultimate objectives for managing runoff from the site. The objectives in nonpoint source pollution and stormwater management have gradually evolved over the years, and may vary considerably among jurisdictions. However, the local planner and engineering consultant often do recognize several common and general goals which should be incorporated into a BMP plan. At a minimum, the BMP plan jointly developed for a site should accomplish the following goals:

- Reproduce, as nearly as possible, the hydrological conditions in the stream prior to development.
- Provide a moderate level of removal for most urban pollutants.
- Be appropriate for the site, given physical constraints.
- Be reasonably cost-effective in comparison with other BMPs.
- Have an acceptable future maintenance burden.
- Have a neutral impact on the natural and human environment.

Reproduce Pre-development Hydrological Conditions

The historical concern in stormwater management has been to reduce the frequency and severity of downstream floods. In most areas, this goal is achieved by controlling the peak discharge computed for a specific design storm to pre-development levels. In reality, however, floods are but one of a series of hydrological changes brought about by watershed development. Other hydrological changes can have equally profound impacts on the quality of downstream aquatic habitat and/or the severity of streambank erosion. Some BMP options are capable of mitigating these impacts through artificial groundwater recharge or the control of small to intermediate storm events. Both the planner and the engineer should check the condition of stream channels downstream to determine if such options should be required.

Provide Moderate Pollutant Removal Capability

In recent years, BMP designs have been adapted to enhance pollutant removal during storms, and thereby, improve the quality of stormwater runoff delivered to receiving waters. BMPs differ markedly in the pollutant removal mechanisms they employ, and consequently, their performance in removing different pollutants can vary significantly. However, the engineer has some ability to enhance removal rates by increasing the volume of runoff effectively treated by the BMP, or by adding extra design features. The planner's responsibility is to provide specific guidance to the engineer on which urban pollutants are to be targeted for removal in the watershed.

Feasibility for the Site

A surprisingly high number of BMPs are constructed on sites for which they are not suitable. As a consequence, some BMPs are often plagued with chronic maintenance problems or nuisance conditions, and in extreme cases, may no longer function as designed. To prevent these sorts of problems from occurring, both the planner and engineer should clearly understand the physical restrictions associated with each BMP. In addition, the engineer should perform field tests to verify the physical condition of the site. Depending on the results, the engineer may have to modify the BMP plan or incorporate preventative design features.

Cost-effectiveness

The construction costs for different BMP options can vary substantially, even on similar sites. This is due to inherent differences in the methods and materials used for BMPs, as well as certain economies-of-scale. Since BMP costs are eventually passed on to the consumer, cost-minimization should be a priority for both the engineering consultant and the planner. Generally, the engineering consultant, who has a legitimate interest in developing the least cost plan for his or her client, will perform the needed cost analyses.

Acceptable Future Maintenance Burden

Like any other pollution control device, BMPs can only continue to be effective if they are regularly inspected and maintained. Maintenance tasks for most BMPs include both low cost routine tasks and more expensive non-routine tasks, such as rehabilitation or sediment removal. Maintenance costs for BMPs are significant. Over a twenty year interval they will often equal or exceed the initial construction cost. However, the cost and responsibility for maintenance is normally passed on to future residents or the public sector, and not the original developer.

Consequently, the planner must clearly vest responsibility for maintenance: How and when tasks will be performed, how it is to be financed, and who will inspect the BMP. In most cases, the maintenance burden of a BMP is ultimately rooted in the initial design and construction of the facility. Planners and engineers should work together in this phase to anticipate future maintenance problems at the site and develop designs that can alleviate them. If maintenance requirements are addressed during the design and construction phases, both the scope and cost of future maintenance activities can be sharply reduced.

Neutral Impact on the Environment

Urban BMPs nearly always represent a significant modification to both the natural environment and the adjacent community. As such, BMPs can either enhance or degrade the amenity values that both provide. Comparatively small investments in design, landscaping and maintenance can make a BMP an attractive feature of a community, or at least an unobtrusive one. Without such efforts, many BMPs become "dead space" in a development; that is, they appear unsightly or discordant, provide no habitat or recreational opportunities, and are plagued by nuisance problems. The importance of enhancing the amenity values of a BMP cannot be overemphasized, as resident perceptions about a BMP are generally formed by the amenities they do or do not provide. These perceptions, in turn, strongly influence their acceptance of and support for BMPs, which is critical if the same residents are expected to pay for maintenance.

BMP SCREENING TOOLS

To aid the planner or engineer in choosing the best BMP for a site, a series of screening tools have been developed to compare the capabilities and limitations of each BMP. The first screening tool can be used to identify which BMP options are physically feasible for the site (Figures 2.1 and 2.2). These help the designer to shorten the list of BMP options that need be considered at a site. The second screening tool (Figure 2.3) summarizes the stormwater benefits which are provided by each BMP option. The third screening tool (Figure 2.4) provides rapid guidance on the relative pollutant removal capability of BMPs for a number of urban pollutants of concern. Finally, the fourth screening tool (Figure 2.5) indicates what natural or human amenities, if any, can be provided by the BMP.

Assuming that one or more BMP option has been selected, final design can then begin. More detailed information on the specific design methods, maintenance requirements and cost estimating techniques for the BMP option can be found using the summary index (Table 2.1). The reader is also encouraged to refer to each individual BMP chapter during all phases of the design.

SCREENING BMPs BASED ON PHYSICAL SUITABILITY

The first step in choosing a BMP is to identify which BMPs are actually suitable for physical conditions of the site. The two most important physical factors to consider in this assessment are the total contributing watershed area, and the infiltration rate of the soils of the site. Most BMPs can only be applied within relatively narrow ranges of watershed area and soil types. Figure 2.1 shows these ranges in schematic fashion. Solid black bars indicate when these two factors do not pose a problem, and the absence of a bar denotes that the BMP probably should not be applied under the specified condition. In cases where the bar becomes narrow, the BMP may or may not be feasible for the site depending on local design standards, development intensity, or the expected level of future maintenance.

Figure 2.2 presents a matrix that shows whether a BMP is also subject to other physical restrictions. In these cases, a solid dot indicates that the factor is not normally a restriction, whereas an open dot suggests that it is a restriction. In most cases, these restrictions do not necessarily prevent the use of a BMP option, but may affect where a BMP is located on a site, or how it is designed. As a general rule, pond BMPs normally face fewer of these site restrictions than infiltration BMPs.

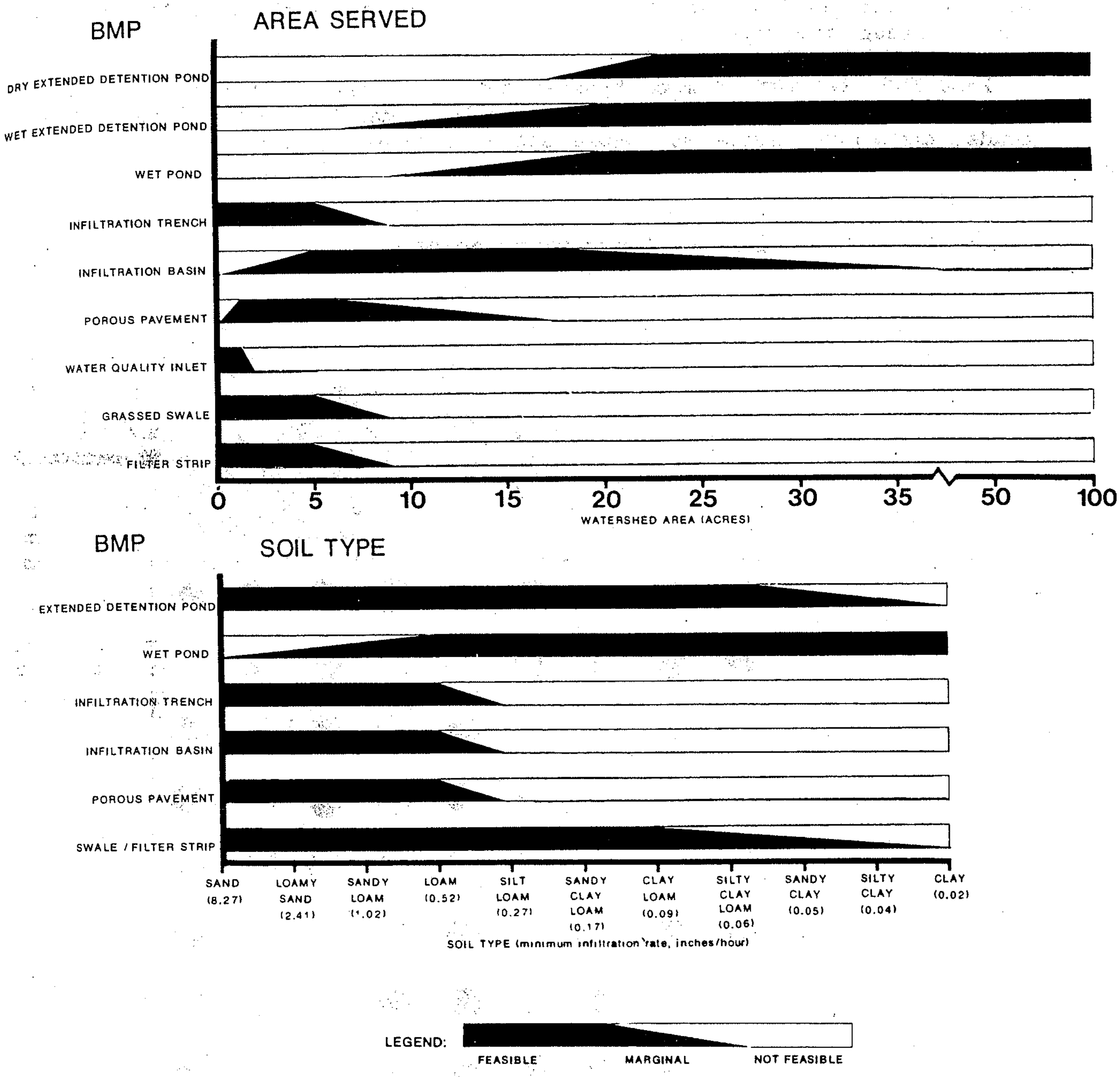
The nature of each of the physical factors outlined in these screening tools are described below.

Watershed Area Served

Pond BMPs normally require a significant contributing watershed area (greater than ten acres) to ensure proper operation. The lower range of suitability for ponds is set by the minimum orifice size for dry extended detention ponds, or the capacity to maintain water levels in wet ponds and wet extended detention ponds. By way of contrast, infiltration and vegetative BMPs are generally only applicable on sites less than ten acres, due to space, economic or flow velocity constraints.

It should be noted that the contributing area of a site does not always have to be fixed. By creatively using local topography and drainage, site area can be increased or decreased to better accommodate a particular BMP. For example, additional runoff generated away from the site (off-site runoff) can be routed to the BMP, thereby increasing total site area and making pond options more feasible. Conversely, various portions of the total runoff from a site can be routed to individual BMPs (decreasing site area, and making infiltration and vegetative BMPs more practical).

Figure 2.1: Watershed Area and Soil Permeability Restrictions for BMPs



Soil Type

The permeability of the soil underlying a BMP has a profound influence on its effectiveness. This is particularly true for infiltration BMPs, which cannot be applied on sites with soils that have infiltration rates (fc) less than 0.27 inches/hour, as defined by the least permeable layer in the soil profile. This excludes most "C" and "D" soils which cannot exfiltrate enough runoff through the subsoil.

Pond BMPs tolerate a much broader range of soil conditions. Extremely permeable sandy soils may make it difficult to maintain water levels in wet ponds, and clayey soils may cause standing water problems in dry extended detention ponds.

Figure 2.2: Other Common Restrictions on BMPs

BMP	SLOPE	HIGH WATER TABLE	CLOSE TO BEDROCK	PROXIMITY TO FOUNDATIONS	SPACE CONSUMPTION	MAXIMUM DEPTH	RESTRICTED LAND USES	HIGH SEDIMENT INPUT	THERMAL IMPACTS
EXTENDED DETENTION POND	●	●	◑	●	○	●	●	◑	●
WET POND	●	●	◑	●	○	○	●	◑	○
INFILTRATION TRENCH	○	○	○	○	●	○	●	○	●
INFILTRATION BASIN	◑	○	○	◑	◑	○	●	○	●
POROUS PAVEMENT	○	○	○	○	○	○	○	○	●
WATER QUALITY INLET	●	●	○	○	●	○	○	○	●
GRASSED SWALE	○	○	◑	◑	●	●	○	○	●
FILTER STRIP	◑	◑	◑	◑	●	●	◑	○	●

○ MAY PRECLUDE THE USE OF A BMP
◑ CAN BE OVERCOME w/CAREFUL SITE DESIGN
● GENERALLY NOT A RESTRICTION

Slope

Steep slopes restrict the use of several BMPs. For example, porous pavement and grassed swales must be situated in sites with slopes of 5% or less. Also, infiltration trenches and filter strips are not practical when slopes exceed 20%.

High Water Table

The water table acts as an effective barrier to exfiltration and can sharply reduce the ability of an infiltration BMP to drain properly. If the height of the seasonally high water table extends to within four feet of the bottom of an infiltration BMP, the site is seldom considered suitable.

Close to Bedrock

The downward exfiltration of storm runoff is also impeded if the bedrock layer lies too close to the soil surface. As with a high water table, a close bedrock layer prevents an infiltration BMP from draining properly. Therefore, if the bedrock layer extends to within 2 to 4 feet of the bottom of an infiltration BMP, the site is not feasible. Similarly, pond BMPs are often not feasible if bedrock lies within the area that must be excavated to provide stormwater storage.

Proximity to Foundations and Wells

Since infiltration BMPs divert runoff back into the soil, some sites may experience problems with local seepage. This can be a real problem if the BMP is located too close to a building foundation. Another risk is that the runoff and pollutants diverted into the groundwater may contaminate water supplies. While relatively little research has been performed to evaluate this risk, it is advisable to keep infiltration BMPs located at least 100 feet away from drinking water wells.

Land Consumption

Some sites are so small or so intensively developed that no room is available for BMP options that consume a large amount of space. Pond BMPs and porous pavement both require a large surface area and a generous buffer, and consequently may not fit into extremely tight sites.

Maximum Depth

To preserve storage capacity and provide optimal pollutant removal conditions, infiltration BMPs must be designed to completely drain within 2 to 3 days after a storm. If the infiltration rates of the underlying soils are marginal, the depth of the infiltration facility may be limited. These restrictions vary depending on whether the facility is a trench, basin or porous pavement facility.

Wet ponds are also subject to a maximum depth limit as well. Extremely deep ponds (greater than 8 feet deep) may stratify during the summer and create low oxygen conditions near the bottom of the pond. This in turn, creates the potential for the release of pollutants from the sediments back into the water column.

Restricted Land Uses

Certain BMPs can only be applied to particular land uses, and are not broadly applicable for all development sites. Porous pavement, for example, can only be used for sites with parking lots not expected to receive heavy car or truck traffic. Similarly, grassed swales can only be used in conjunction with low density residential areas or roads.

High Sediment Input

Most BMPs are unable to handle the large loads of sediment eroded during the construction phase of development. Infiltration BMPs are particularly susceptible to rapid clogging and subsequent failure if significant sediment loads are allowed to enter the structure. As a general rule, these BMPs should not be installed until all of the land disturbed by construction in the contributing watershed is effectively stabilized. Contractors must often take unusual steps during the actual installation of the infiltration BMPs to prevent soil compaction or sediment contamination. Although sediment loads drop sharply after the construction phase, gradual clogging of infiltration BMPs can still occur, so many designs call for the use of a pre-treatment device to filter out sediment and other coarse particles before they reach the facility.

Pond BMPs can be used for sediment control during the construction phase of development, with proper conversion, clean-out and regrading. After the site is stabilized, significant amounts of pond storage capacity can still be lost due to the gradual accumulation of deposited sediments. After 5 to 20 years, the sediment deposits are large enough to impair the function of a pond, and must be removed. The cost and scope of sediment removal can be reduced by preventative design, extra storage and/or sediment forebays.

Thermal Enhancement

Shallow marshes and wet ponds warm up rapidly during the summer months. Under certain circumstances, runoff leaving these BMPs can be 5 to 10 degrees warmer than the runoff entering the structure. Such warm water release can be a lethal thermal shock to aquatic organisms that are adapted to coldwater conditions. Thus, the use of wet ponds and shallow marshes should be avoided in watersheds with sensitive coldwater streams.

SCREENING BMPs BASED ON STORMWATER BENEFITS PROVIDED

The objective of stormwater management is to attempt to reproduce the pre-development hydrology of the site. As noted earlier, this can be done through a combination of peak discharge control, volume control, groundwater recharge and streambank erosion control. Figure 2.3 shows the extent to which common BMP designs provide these benefits. A solid dot indicates that the BMP normally provides the benefit; an open dot indicates that it does not; and a half dot suggests that the benefit might be provided in certain sites or with special design modifications. As can be seen, very few BMP options can achieve the full spectrum of desirable stormwater benefits. This is because a different flow condition and/or frequency must be controlled to provide each benefit. As an example, the designer needs to control very large, infrequent storms to attain peak discharge control, yet must concentrate on much smaller and more frequent storms to provide groundwater recharge.

The design variations for infiltration BMPs shown in Figure 2.3 deserve some additional explanation. The term "exfiltration" refers to the amount of runoff that is effectively infiltrated through the soil profile. Full exfiltration occurs when all of the runoff delivered to an infiltration BMP is completely exfiltrated back into the soil. As one might imagine, full exfiltration BMPs need to be very large in volume. Partial exfiltration BMPs only divert a fixed volume of runoff into the soil (the remaining runoff is conveyed through the BMP, but may be detained long enough to provide some peak discharge control). In water quality exfiltration BMPs, a small, fixed runoff volume is diverted into the soil. The remaining runoff is conveyed away, and is not detained long enough to provide any peak discharge control.

Peak Discharge Control

As shown in Figure 2.3, peak discharge control is often required for one or more design storms under local regulations. The most common design storm used is the 2 year storm, which is a flood that occurs, on average, every two years. In natural watersheds, the two year storm produces a flood that fills a stream to the top of its banks (i.e., the bankfull flood). Some jurisdictions also require control of the 10 or 100 year design storms, particularly if there is unprotected development further downstream on the floodplain. Even if a BMP does not control these larger design storms, they must still be designed to safely pass them through (e.g., using an emergency spillway or an overflow pipe).

Peak discharge control is accomplished in pond BMPs by temporarily detaining a large portion of the runoff volume for the design storm, and then releasing it at the lower pre-development rate. This is done by using a vertical riser with a control orifice or weir. A single pond can control a series of design storms by using a series of orifices and weirs at progressively higher elevations. In general, pond BMPs are an excellent means of providing peak discharge control.

Infiltration BMPs have a more limited capacity to control peak discharges. Full exfiltration systems are normally only capable of controlling peak discharges for the 2 year storm (and in rare cases, the 10 year storm). Most partial exfiltration systems can control the 2 and 10 year storm, and pass the 100 year storm. Water quality exfiltration systems, water quality inlets, swales, and filter strips normally have little or no capacity to control peak discharges.

Volume Control

Infiltration BMPs can help to reduce the increased runoff volumes generated from small and intermediate storms, since they divert a significant fraction of storm runoff volume back into the soil. Pond BMPs, on the other hand, are ineffective in reducing runoff volume. Ponds only detain or retain runoff for a short period of time before releasing it downstream.

Figure 2.3: Comparative Stormwater Benefits Provided by Urban BMPs

BMP	PEAK DISCHARGE CONTROL					
	2 YEAR STORM	10 YEAR STORM	100 YEAR STORM	VOLUME CONTROL	GROUNDWATER RECHARGE	STREAMBANK EROSION CONTROL
EXTENDED DETENTION						
DRY	●	●	●	○	○	●
DRY w/ MARSH	●	●	●	◑	○	●
WET	●	●	●	○	○	●
WET POND	●	●	●	◑	○	○
INFILTRATION TRENCH						
FULL EXFILTRATION	●	◑	○	●	●	●
PARTIAL EXFILTRATION	●	●	○	●	●	●
WATER QUALITY TRENCH	○	○	○	●	●	●
INFILTRATION BASIN						
FULL EXFILTRATION	●	◑	○	●	●	●
INFILTRATION/DETENTION	●	●	●	●	●	◑
OFF-LINE BASIN	○	○	○	●	●	◑
POROUS PAVEMENT						
FULL EXFILTRATION	●	◑	○	●	●	●
PARTIAL EXFILTRATION	●	●	◑	●	●	◑
WATER QUALITY	○	○	○	●	●	◑
WATER QUALITY INLET	○	○	○	○	○	○
GRASSED SWALE	◑	○	○	◑	◑	○
FILTER STRIP	◑	○	○	◑	◑	○

○ SELDOM OR NEVER PROVIDED

◑ SOMETIMES PROVIDED w/ CAREFUL DESIGN

● USUALLY PROVIDED

Groundwater Recharge

Infiltration BMPs are an excellent means of providing for groundwater recharge, which is often lost as a consequence of watershed development. "Natural" levels of groundwater recharge can be duplicated by diverting a significant fraction of the runoff from frequent small and moderate storms back into the soils. Most exfiltration designs recharge the groundwater sufficiently to sustain normal low flows in headwater streams during the critical summer months. Vegetative BMPs, such as grassed swales and filter strips have a more limited capability, and pond BMPs generally have little or none.

Streambank Erosion Control

All BMPs that control peak discharges for the 2 year storm provide some degree of streambank erosion control. However, the 2 year storm creates an erosive condition in natural channels (i.e., a bankfull discharge). To adequately protect downstream channels, it is necessary to control both the post-development increase in the 2 year bankfull flood and the increased frequency with which it occurs. This normally entails the control of storm events of intermediate size (less than the 2 year storm and greater than the mean storm). Some preliminary design suggestions for minimizing the increased frequency of bankfull flooding are provided in Appendix B. Based on this analysis, it appears that extended detention ponds and some infiltration BMPs can effectively reduce the frequency with which bankfull flooding occurs, if sized properly. Wet ponds (without extended detention), vegetative BMPs, and water quality inlets show little capability in this regard.

SCREENING BMPs BASED ON POLLUTANT REMOVAL BENEFITS

The pollutant removal capability of a BMP is primarily governed by three interrelated factors: 1) the removal mechanisms used, 2) the fraction of the annual runoff volume that is effectively treated, and 3) the nature of the urban pollutant being removed. The designer has a limited ability to control the first two factors, but has no influence on the third.

Figure 2.4 illustrates the comparative pollutant removal capabilities of BMP options. The removal rates shown are inferred from field performance monitoring, laboratory experiments, modeling analyses and theoretical considerations. Due to the inherent uncertainties involved, removal rates are expressed in 20 percent increments. A removal rate has been estimated for several design variations of each BMP. The design variations for each BMP are arrayed in order of increasing fractions of annual runoff volume treated.

As noted earlier, the nature of the pollutant being removed often sets an upper limit on the potential removal rate that can be achieved. From an operational standpoint, pollutants can be said to exist in either particulate or soluble forms, or more commonly, as a mix of both forms (MWCOG, 1987). Particulate pollutants, such as sediment and lead, are relatively easy to remove by common BMP removal mechanisms, including settling and filtering. Soluble pollutants, such as nitrate, phosphate, and some trace metals, are much more difficult to remove. Settling and filtering removal mechanisms have little or no effect, and biological mechanisms, such as uptake by bacteria, algae, rooted aquatic plants or terrestrial vegetation, must be used.

The importance of pollutant form can clearly be seen in Figure 2.4. Most BMPs can achieve an extremely high removal rate for suspended sediment and trace metals that exist largely in particulate forms. Much lower removal rates are generally obtained for total phosphorus, oxygen-demanding materials and total nitrogen, since they typically exist as a mix of particulate and soluble forms.

The following sections summarize the pollutant removal capability of BMPs, with an emphasis on their major removal mechanisms, and design enhancements which can be used to improve their performance.

Extended Detention Ponds

Dry extended detention ponds rely primarily on settling to remove pollutants. Depending on how much and how long runoff is detained, it is possible to achieve moderate or high removal rates for particulate pollutants that are relatively easy to settle. However, removal rates for most soluble pollutants are quite low for dry extended detention ponds, although it is possible to enhance rates by incorporating biological removal mechanisms into the design of the pond (e.g., by establishing a shallow marsh in the bottom stage of a dry extended detention pond, or by using extended detention in combination with a wet pond).

Wet Ponds

Wet ponds have a moderate to high capability of removing most urban pollutants, depending on how large the volume of the permanent pool is in relation to the runoff produced from the surrounding watershed. Wet ponds utilize both settling and biological uptake, and are capable of removing both particulate and soluble pollutants. In addition to increasing the volume of the permanent pool, wet pond removal rates can be enhanced by establishing marshes around the perimeter, and by adjusting the geometry of the pond.

Infiltration Practices (trenches, basins, porous pavement)

From a pollutant removal standpoint, infiltration trenches, basins, and porous pavement behave in a similar manner, and can be treated as a group. Infiltration practices filter runoff through the soil layer, where a number of physical, chemical and biological removal processes occur. Infiltration practices have a moderate to high removal capability for both particulate and soluble urban pollutants, depending how much of the annual runoff volume is effectively exfiltrated through the soil layer. Removal rates can be further enhanced by increasing the surface area reserved for exfiltration and adjusting the geometry of the practice to achieve a draining time of less than 3 days. It should be noted that infiltration practices should <u>not</u> be relied on to achieve high levels of particulate pollutant removal (particularly sediments), since these particles can rapidly clog the device. Rather, particulate pollutants should be removed before they enter the structure by means of a filter strip, sediment trap or other pretreatment device.

Figure 2.4: Comparative Pollutant Removal Of Urban BMP Designs

BMP/design	SUSPENDED SEDIMENT	TOTAL PHOSPHORUS	TOTAL NITROGEN	OXYGEN DEMAND	TRACE METALS	BACTERIA	OVERALL REMOVAL CAPABILITY
EXTENDED DETENTION POND							
DESIGN 1	◕	◔	◔	◔	◑	⊗	MODERATE
DESIGN 2	●	◑	◔	◑	◕	⊗	MODERATE
DESIGN 3	●	◕	◑	◑	◕	⊗	HIGH
WET POND							
DESIGN 4	◕	◑	◔	◔	◔	⊗	MODERATE
DESIGN 5	◕	◑	◔	◔	◕	⊗	MODERATE
DESIGN 6	●	◕	◑	◑	◕	⊗	HIGH
INFILTRATION TRENCH							
DESIGN 7	◕	◑	◑	◕	◕	◕	MODERATE
DESIGN 8	●	◑	◑	◕	●	◕	HIGH
DESIGN 9	●	◕	◕	●	●	●	HIGH
INFILTRATION BASIN							
DESIGN 7	◕	◑	◑	◕	◑	◕	MODERATE
DESIGN 8	●	◑	◑	◕	●	◕	HIGH
DESIGN 9	●	◕	◕	●	●	●	HIGH
POROUS PAVEMENT							
DESIGN 7	◑	◕	◑	◕	◑	◕	MODERATE
DESIGN 8	●	◕	◕	◕	●	●	HIGH
DESIGN 9	●	◕	◕	●	●	●	HIGH
WATER QUALITY INLET							
DESIGN 10	○	⊗	⊗	⊗	⊗	⊗	LOW
FILTER STRIP							
DESIGN 11	◔	○	○	○	◔	⊗	LOW
DESIGN 12	●	◑	◑	◕	●	⊗	MODERATE
GRASSED SWALE							
DESIGN 13	○	○	○	○	○	⊗	LOW
DESIGN 14	◔	◔	◔	◔	○	⊗	LOW

KEY:

○	0 TO 20% REMOVAL
◔	20 TO 40% REMOVAL
◑	40 TO 60% REMOVAL
◕	60 TO 80% REMOVAL
●	80 TO 100% REMOVAL
⊗	INSUFFICIENT KNOWLEDGE

Design 1: First-flush runoff volume detained for 6-12 hours.
Design 2: Runoff volume produced by 1.0 inch, detained 24 hours.
Design 3: As in Design 2, but with shallow marsh in bottom stage.
Design 4: Permanent pool equal to 0.5 inch storage per impervious acre.
Design 5: Permanent pool equal to 2.5 (Vr); where Vr=mean storm runoff.
Design 6: Permanent pool equal to 4.0 (Vr); approx. 2 weeks retention.
Design 7: Facility exfiltrates first-flush; 0.5 inch runoff/imper. acre.
Design 8: Facility exfiltrates one inch runoff volume per imper. acre.
Design 9: Facility exfiltrates all runoff, up to the 2 year design storm.
Design 10: 400 cubic feet wet storage per impervious acre.
Design 11: 20 foot wide turf strip.
Design 12: 100 foot wide forested strip, with level spreader.
Design 13: High slope swales, with no check dams.
Design 14: Low gradient swales with check dams.

Water Quality Inlets

Current designs of water quality inlets appear to have low to moderate removal rates for particulate pollutants, and low to zero rates for soluble pollutants. Water quality inlets rely primarily on settling for removal, and given their small storage capacity and brief residence times, it is likely that only coarse grit, sand, and some silts will be trapped. Inlets do show some promise in removing hydrocarbons, such as oil, gas and grease, from runoff. Due to resuspension problems, however, pollutant removal can only be attained in water quality inlets if they are cleaned regularly.

Filter Strips

Filter strips have a low to moderate capability of removing pollutants in urban runoff, and exhibit higher removal rates for particulate rather than soluble pollutants. Removal mechanisms include filtering (through vegetation and/or soil), settling/deposition and uptake by vegetation. Forested buffer strips appear to have a higher removal capability than grass buffer strips. However, length, slope and soil permeability are critical factors which influence the effectiveness of any strip. Another practical design problem is how to prevent runoff from concentrating and thereby "short-circuiting" the strip. Special design modifications and regular maintenance are needed to provide optimal removal rates in the field.

Grassed Swales

Grassed swales have a low capability of removing urban pollutants, except under site conditions which are unusual in the Washington metropolitan area (e.g., extremely gentle slopes, permeable and uncompacted soils, installation of check dams and maintenance of a dense grass turf). If constructed under these conditions, pollutants can be removed through the filtering action of the grass, by deposition in low velocity areas, and by exfiltration through the soil layer. Moderate removal of particulate pollutants, and low removal of soluble pollutants can be expected under these optimal conditions.

SCREENING BMPs FOR ENVIRONMENTAL AMENITIES

Figure 2.5 is a screening tool that shows the environmental and human amenities which can be provided by a particular BMP. In most cases, these amenities are not automatically provided when a BMP is built. Rather, they are a result of thoughtful design, regular maintenance, and creative landscape planting. In this matrix, a solid dot indicates that there is a strong potential for a BMP to provide the amenity; an open dot indicates the BMP has little or no potential; and a half dots suggests that a BMP might provide the amenity with some design modifications or as a result of unusual site conditions.

The first five headings in Figure 2.5 refer to amenities related to the improvement of the natural environment, while the last five headings pertain to amenities which are provided to the adjacent community. As might be expected, community amenities are quite subjective, and often adjacent residents hold widely divergent opinions as to their value. However, based on opinion surveys and less formal surveys of complaints to public works officials, some generalities have been made.

Figure 2.5: Environmental and Community Amenities Provided by BMPs

BMP	LOW FLOW MAINTENANCE	STREAMBANK EROSION CONTROL	AQUATIC HABITAT CREATION	WILDLIFE HABITAT CREATION	NO THERMAL ENHANCEMENT	LANDSCAPE ENHANCEMENT	RECREATIONAL BENEFITS	HAZARD REDUCTION	AESTHETICS	COMMUNITY ACCEPTANCE
RY EXTENDED DETENTION	○	●	◑	●	●	◑	◑	◑	◑	◑
DED DETENTION w/ MARSH	○	●	●	●	○	◑	○	◑	◑	◑
ET EXTENDED DETENTION	○	●	●	●	○	●	●	◑	◑	●
WET POND	○	○	●	●	○	●	●	◑	◑	●
INFILTRATION TRENCH	●	◑	○	○	●	○	○	●	○	●
INFILTRATION BASIN	●	◑	○	●	●	◑	◑	●	○	◑
POROUS PAVEMENT	●	◑	○	○	●	○	○	●	○	●
WATER QUALITY INLET	○	○	○	○	●	○	○	●	○	●
GRASSED SWALE	◑	○	○	◑	●	◑	○	●	◑	●
FILTER STRIP	◑	○	○	●	●	◑	○	●	◑	●
SHALLOW MARSH	○	○	●	●	○	◑	○	◑	◑	◑

○ SELDOM PROVIDED

◑ SOMETIMES PROVIDED (w/ Design Modifications)

● USUALLY PROVIDED

Low Flow Maintenance

Downstream aquatic life can be jeopardized when the natural low flow levels experienced during the summer months decline even further because of reduced infiltration in urbanized watersheds. Infiltration BMPs contribute significantly to groundwater recharge and appear to be capable of sustaining low flows during the critical summer months if widely applied in a watershed. Vegetative BMPs, such as swales and filter strips, appear to have modest potential in this regard, and pond BMPs have little effect in maintaining low flows.

Streambank Erosion Control

Streambank erosion not only contributes large sediment loads to receiving waters, but also has an adverse impact on the habitat quality for downstream aquatic life. Some BMPs, such as extended detention ponds and full exfiltration BMPs, can control erosive stormflows enough to keep downstream channels and banks relatively stable, whereas most other BMPs have only marginal capabilities.

Aquatic Habitat Creation

Some BMP options are attractive in that they can create wetland or open water areas utilized by waterfowl, marsh birds, and other wildlife. Shallow marshes and wet ponds are particularly well suited for this role, if relatively small investments are made in landscaping design and plant selection. "Volunteer" wetland plants may also colonize these BMPs (and poorly drained extended detention ponds) without intentional planting efforts, but may not provide high quality habitat. Tips for enhancing aquatic habitat are presented in the Chapter 4 and the Basin Landscaping Guide (Chapter 9).

Wildlife Habitat Creation

BMPs with generous buffers (wet ponds, extended detention ponds, infiltration basins and filter strips) present good opportunities for creating terrestrial wildlife habitat. The buffer areas (and sometimes the basin floors) can be managed as wet meadows, thus reducing mowing costs for the facility. Relatively diverse biological communities can be further enhanced through judicious planting of trees, shrubs and grasses that provide food and cover for wildlife (see Chapter 9). These communities have added value because of the general scarcity of wildlife habitat in urbanized areas.

No Thermal Enhancement

As noted earlier, wet ponds can be detrimental in some watersheds as they heat water passing through the structure during the summer months. Their use is often restricted in watersheds that contain sensitive coldwater fisheries, such as those that support native trout populations.

Landscape Enhancement

Few BMPs will be an attractive feature of a community unless serious efforts are directed toward natural grading, landscaping and regular maintenance. If properly designed, pond options probably have the most potential to enhance the urban landscape. Wet ponds are frequently used to create a waterfront effect in residential developments, and may actually increase the value of adjacent property. Vegetative BMPs have a less dramatic effect on landscape values, and most infiltration BMPs and dry extended detention ponds have a neutral or negative effect.

Recreational Benefits

With the exception of large wet ponds, few BMPs provide active recreational opportunities (e.g., fishing, swimming, or skating). In fact most jurisdictions generally do not encourage such activities, as they may invite vandalism or liability problems. However, if properly landscaped, pond BMP options can provide passive recreation opportunities for adjacent residents, such as walking, birdwatching, or nature enjoyment, particularly when combined with bike or jogging paths, picnic areas, and tot-lots situated in nearby open space. In rare instances, the floors of extended detention ponds can even be used for ballfields and play areas.

Hazard Reduction

Careful design of pond BMPs is needed to reduce potential safety hazards. Plans should be analyzed to eliminate obvious hazards, such as steep side-slopes, deep water, sudden drop-offs from the shore, or dangerous outlet/pipe configurations. Most infiltration BMPs entail little if any safety risks, and some (porous pavement) are thought to reduce certain traffic safety problems.

Aesthetic Value

As shown in Figure 2.5, most pond options have the potential to be either an attractive or an unattractive feature of a community, depending on the attention paid to their design, landscaping ,and maintenance. Artificial contours should be avoided, and control structures (risers, low flow channels, outlets and riprap) should be concealed in the embankment, or by vegetation where feasible. Infiltration BMPs generally have little potential to be attractive, but can at least be designed to be unobtrusive.

Community Acceptance

Surveys of resident perceptions about adjacent BMPs have revealed that most BMPs are acceptable if regular cosmetic maintenance is performed. Residents often indicate a preference for wet ponds over dry ponds. Their response to infiltration BMPs is not well documented. Residents' primary concerns often center around perceived nuisance conditions (algae blooms, odors, mosquitos, weeds, trash, turbidity, etc.), most of which are temporary conditions which should seldom occur if the BMP is properly designed and maintained.

HOW TO USE THE SCREENING TOOLS

The following examples provide illustrations of how the screening tools provided in this Chapter are best used.

EXAMPLE 2-1:USE OF THE SCREENING TOOLS

A developer has a 5 acre parcel which will be converted into an office complex and parking lot. The site has low slopes and permeable, sandy loam soils. The management objectives for the site are; 1) 2 year peak discharge control, 2) groundwater recharge, and 3) moderate to high removal of all urban pollutants.

Step 1. Using Figure 2.1 the following BMPs are suitable, given the area and soil characteristics of the site:

1. Infiltration Trench
2. Infiltration Basin
3. Porous Pavement
4. Grassed Swale
5. Filter Strip

After checking the other site restrictions contained in Figure 2.2, and the accompanying text, it is apparent that grassed swales and filter strips are not appropriate, given the proposed high land-use intensity. The remaining BMP options appear feasible, but must have some kind of pretreatment device to protect them from high sediment inputs.

Step 2. Full or partial exfiltration designs of the infiltration BMP options can provide the desired 2 year peak discharge control and groundwater recharge benefits:

1. Infiltration trench
2. Infiltration basin
3. Porous pavement

Step 3. If the infiltration BMPs are sized according to design rules 8 or 9, high levels of pollutant removal can be expected for all of the remaining options (see Figure 2.4).

Step 4. All of the remaining BMP options provide a similar level of environmental amenities (Figure 2.5).

Step 5. A final BMP option can now be selected on the basis of costs or maintenance requirements. Estimation of both can be found elsewhere in the text using Table 2.1.

EXAMPLE 2-2:USE OF THE SCREENING TOOLS

A developer is planning to build a residential subdivision on a 40 acre parcel of land, with moderate slopes (5%) and sandy clay loam soils. Local ordinances require peak discharge control for the 2 and 10 year design storm. In addition, the planning review agency has specified that the BMP should provide a high level of streambank erosion control, and provide a moderate to high level of nutrient removal. Because of the intended character of the residential area, the developer would like the BMP to provide environmental amenities. Using the screening tools, identify the BMP option(s) which are physically suitable for the site, and determine whether they can provide the desired benefits.

Step 1. From Figure 2.1, it is evident that most infiltration BMPs are not feasible because of watershed area and soil permeability restrictions. The remaining options are:

1. Dry extended detention pond
2. Wet extended detention pond
3. Wet pond

Based on Figure 2.2, which shows other common site restrictions for BMPs, it appears that no other insurmountable limitations exist for the use of the BMPs listed above.

Step 2. Using Figure 2.3, it is evident that all these BMPs can control peak discharges from the 2 and 10 year storm. However, only dry and wet extended detention ponds can provide the desired level of streambank erosion control.

Step 3. From a nutrient removal standpoint, dry extended detention (design 3, with marsh) and wet extended detention (design 6) both have the potential to provide high levels of removal (Figure 2.4).

Step 4. As shown in Figure 2.5, both dry extended detention with marsh, and wet extended detention ponds provide several natural environmental amenities, with proper landscaping and maintenance. These include streambank erosion control, and wildlife and aquatic habitat creation. From the standpoint of community amenities, wet extended detention appears to be preferable, as it gets higher marks for landscaping, recreation, and resident acceptance.

Based on the screening tool, and the stated management objectives for the site, the most appropriate BMP would be a wet extended detention pond. Methods for estimating construction costs and maintenance requirements for the BMP can be found by referring to the appropriate section of this manual, as outlined in Table 2.1.

FINAL DESIGN CONSIDERATIONS

Once a BMP has been selected for the site, more detailed design of the facility can begin. As noted earlier, the designer should consider a number of design features that can enhance pollutant removal, reduce maintenance needs and costs, reduce construction costs, and provide desired environmental and community amenities. Table 2.1 provides a summary index on where such design information can be found within this manual.

Tips for Enhancing Pollutant Removal

Each BMP chapter in this manual contains a section that provides a series of design tips to maximize the pollutant removal capability of a BMP. These guidelines include: ways of adjusting the size and geometry of a BMP to create ideal removal conditions; how vegetation can be effectively used to promote biological removal; how optimum detention/draining times can be achieved; and other means of achieving high pollutant removal requirements.

Table 2.1: Summary Index for BMP Design

	TIPS FOR ENHANCING POLLUTANT REMOVAL	MAINTENANCE REQUIREMENTS FOR THE BMP	TIPS FOR REDUCING MAINTENANCE NEEDS	PROJECTING BMP CONSTRUCTION COSTS	DESIGN VARIATIONS FOR THE BMP	DETAILED DESIGN METHODS FOR THE BMP	DESIGN SUMMARIES
EXTENDED DETENTION	3.14	3.21	3.24	3.20	3.2	3.26	3.27
WET PONDS	4.4	4.13	4.15	4.12	4.4	4.20	4.21
INFILTRATION TRENCH	5.15	5.21	5.23	5.18	5.2	5.25	5.26
INFILTRATION BASIN	6.9	6.14	6.15	6.13	6.2	6.17	6.18
POROUS PAVEMENT	7.10	7.17	7.20	7.13	7.2	7.22	7.23
WATER QUALITY INLET	8.5	8.8	8.8	8.8	8.2	8.8	8.9
GRASSED SWALE	9.3	9.6	9.6	9.6	–	9.10	–
FILTER STRIP	9.7	9.9	9.9	9.9	–	9.10	–
SHALLOW MARSH	–	–	–	9.5	–	9.18	–

Common Maintenance Requirements

Both the regular and non-routine maintenance requirements for each BMP are described in detail elsewhere in this manual. Where data is available, estimates of the costs associated with maintenance are provided, as are inspection schedules and methods. Where construction methods are critical to the successful operation of the BMP, recent specifications on materials and workmanship are also provided.

Tips For Reducing Maintenance Needs

Engineers should make maintenance reduction a major element of every BMP design. A significant amount of future maintenance activities and costs can be eliminated through preventative design. Each BMP chapter includes a series of tips which should be carefully reviewed. These include suggestions on how to reduce maintenance needs by providing access, using long-lasting construction materials, trapping sediment inputs, creative use of the geometry of the BMP to make routine maintenance easier, providing backup drainage systems, and proper stabilization of erosion-prone areas.

Projecting BMP Construction Costs

Each BMP chapter presents a method for computing a rapid planning estimate of the construction costs of a BMP. Unit costs for major construction components are provided for infiltration and pond BMPs (see Table 7.4). Cost equations are also provided for a number of BMPs. The cost-effectiveness of each BMP, in relation to watershed size, development intensity, and other BMP options, is also discussed (see also MWCOG, 1983a; Wiegand et al, 1986).

Design Variations

Each BMP chapter begins with a section that describes several variations in the basic design of a BMP. These designs, drawn from applications around the Washington, D.C. area, illustrate innovative ways to fit a BMP into a particular development site and combine BMPs together to improve performance or minimize maintenance needs.

Design Methods

This manual does not provide step-by-step methods for the hydrological design of BMPs, nor does it give specific models or equations that can be used to accurately determine the necessary storage requirements, geometry and configuration of a BMP at a particular site. These technical aids, used for the final BMP design, are provided by reference at the end of each chapter.

Design Summary

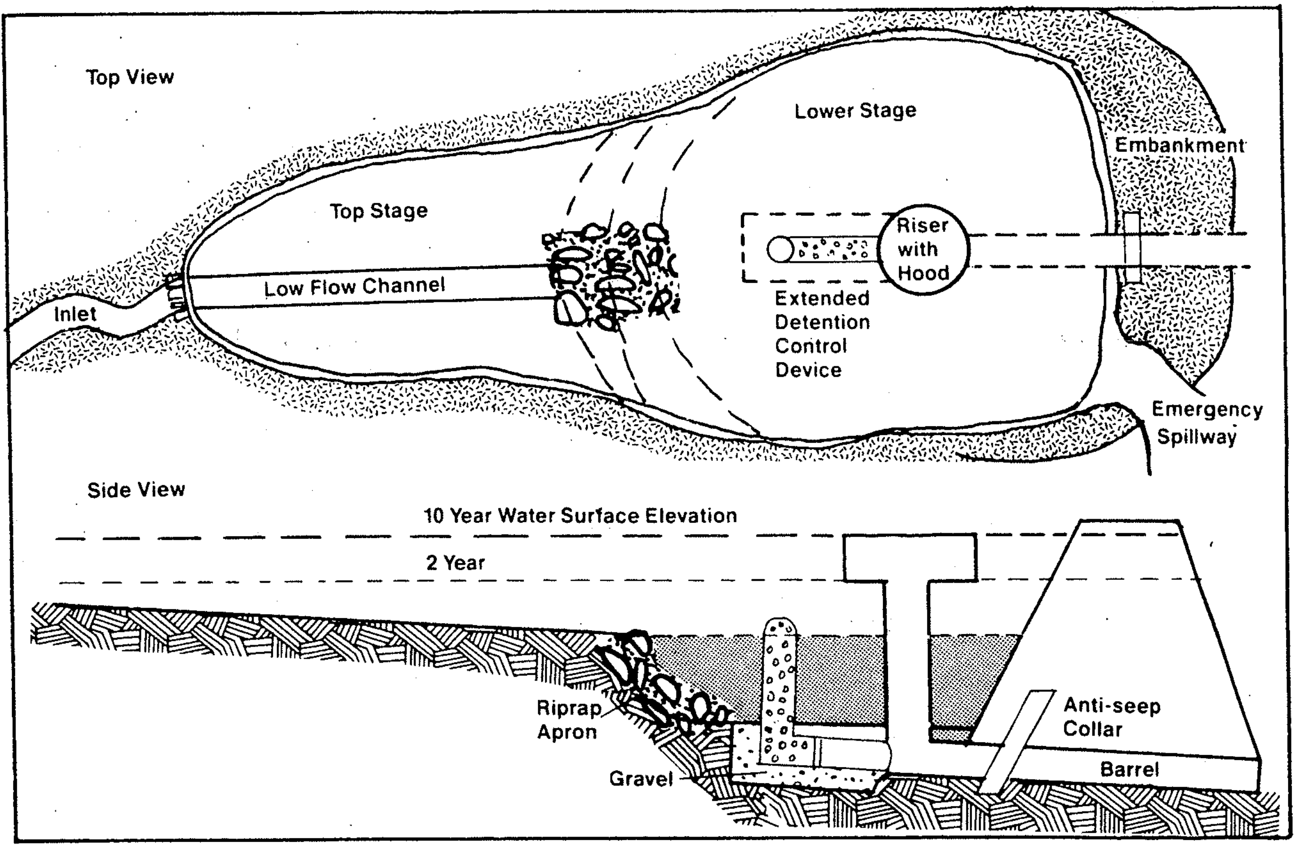
Each chapter concludes with a brief design summary that highlights some of the recommended BMP design features that should be included in every site plan. Both the engineering consultant and the site-plan reviewer can use these summaries as a checklist to ensure that the BMP plan for the site will be effective.

CHAPTER 3: EXTENDED DETENTION PONDS

Extending the detention time of dry or wet ponds is an effective, low cost means of removing particulate pollutants and controlling increases in downstream bank erosion. If stormwater is detained for 24 hours or more, as much as 90% removal of particulate pollutants is possible. However, extended detention only slightly reduces levels of soluble phosphorus and nitrogen found in urban runoff. Removal of these pollutants can be enhanced if the normally inundated area of the pond is managed as a shallow marsh or a permanent pool.

Extended detention ponds significantly reduce the frequency of occurrence of erosive floods downstream, depending on the quantity of stormwater detained and the time over which it is released. Extended detention is extremely cost-effective, with construction costs seldom more than 10% above those reported for conventional dry ponds.

Figure 3.1: Schematic of a Dry Extended Detention Pond



Positive impacts of extended detention ponds include creation of local wetland and wildlife habitat, limited protection of downstream aquatic habitat, and recreational use in the infrequently inundated portion of the pond. Negative impacts include occasional nuisance and aesthetic problems in the inundated portion of the pond (e.g., odor, debris, and weeds), moderate to high routine maintenance requirements, and the eventual need for costly sediment removal. Extended detention generally can be applied in most new development situations, and also is an attractive option for retrofitting existing dry and wet ponds in older urbanized areas.

METHODS USED TO EXTEND DETENTION TIMES

Both wet and dry ponds are easily adapted to achieve extended detention times. A two-stage design is recommended for dry ponds whereby the top portion of the pond is designed to remain dry most of the time, and a smaller portion near the riser is regularly inundated (Figure 3.1). The devices used for extended detention are normally attached to the low flow orifice or the riser. Some frequently used methods to extend detention times in dry and wet ponds are shown in Figures 3.2 and 3.3, respectively, and are described below:

Perforated Riser Enclosed in a Gravel Jacket (dry ponds) [Figure 3.2a]

This experimental design was utilized during the Washington NURP study (MWCOG,1983a). The standard corrugated metal pipe (CMP) riser is perforated with small diameter holes and the normal low flow orifice is closed. The total diameter of all the holes regulates the outflow to achieve the required detention time for all storm events smaller than the two year design storm (which is controlled by the weir on top). A gravel jacket and wire mesh screen are used as a filter to prevent clogging. The perforated riser design has some drawbacks. First, hydraulics of flow through vertical risers are not well defined, which makes it difficult to achieve the target detention time. Second, the bottom portion of the gravel jacket may become clogged over time by deposited sediments.

Perforated Extension of Low Flow Orifice, Inlet Controlled (dry ponds) [Figure 3.2b]

This design is often applied in Northern Virginia (BDET,1986), and entails extending and capping the low flow orifice. Small diameter holes are drilled into the extended PVC pipe, which are protected by 1/4 inch wire mesh, and a layer of gravel and stone. An elbow joint is used to extend the pipe above the surface of the pond to facilitate clean-out operations with high velocity jet hoses. This design should only be considered in areas where regular maintenance clean-outs will be performed, as this device is prone to clogging.

Figure 3.2: Methods For Extending Detention Times in Dry Ponds

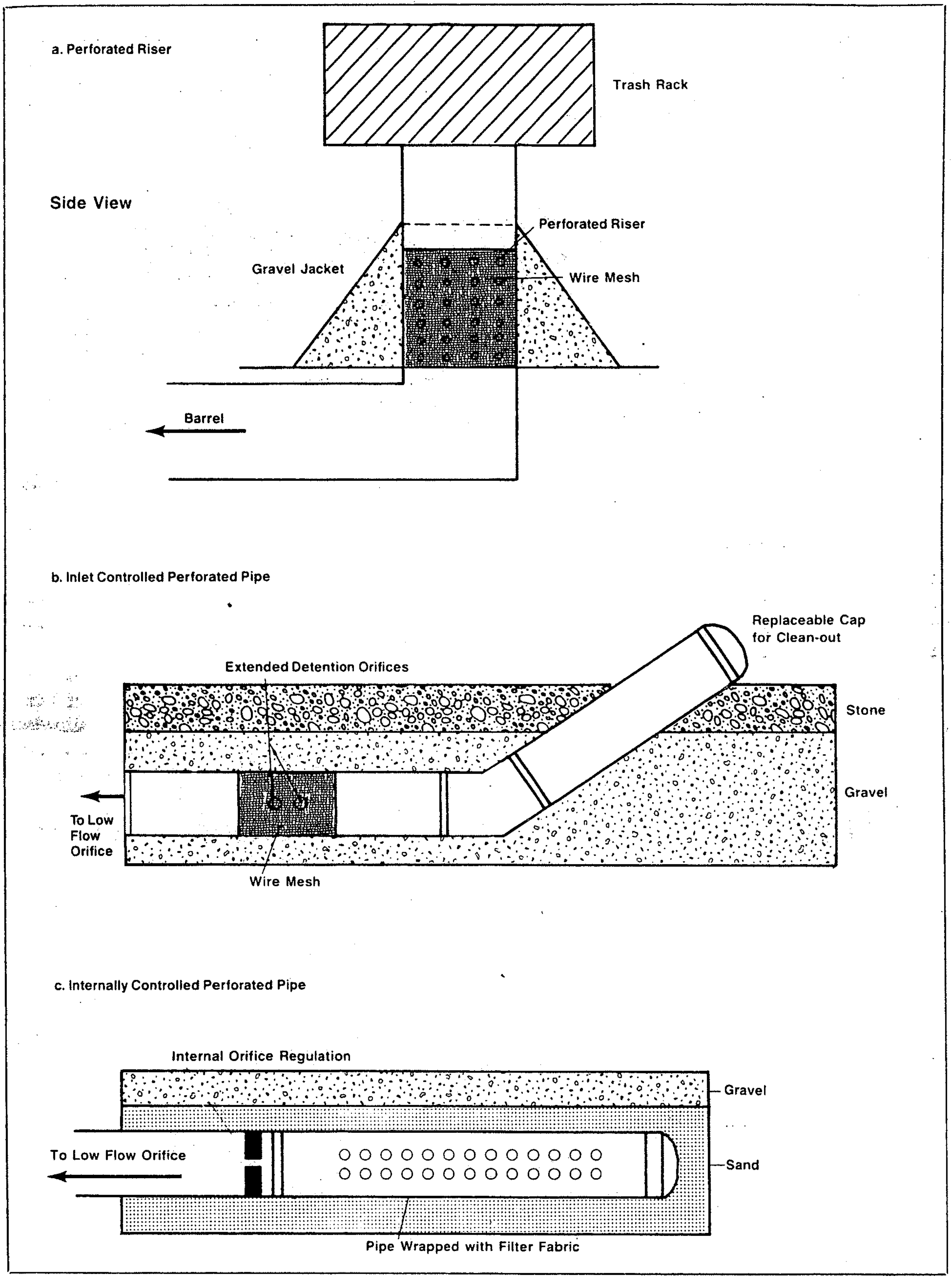
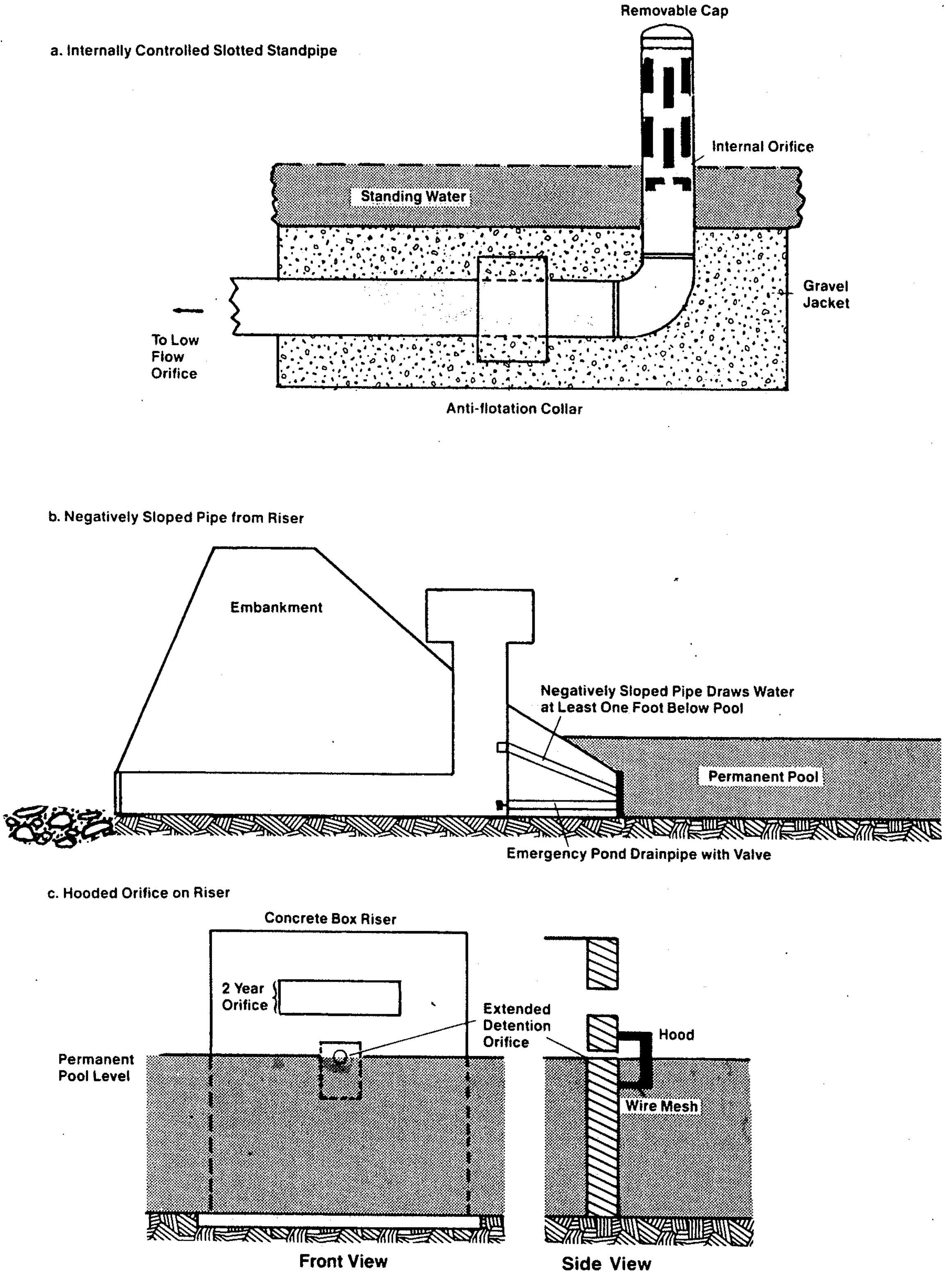


Figure 3.3: Methods For Extending Detention Times In Wet Ponds



Perforated Extension of Low Flow Orifice, Outlet Control (dry ponds) [Figure 3.2c]

Developed by the Baltimore Dept. of Public Works (DPW), this control device also employs a perforated pipe extended from the low flow orifice. The major difference between this design and the previous design is that the release rate of the pipe is regulated by an internal flange within the pipe, rather than by holes drilled through the pipe. This provides additional protection from clogging, as a large number of holes can be drilled on the outward side of the flange. In the event that sediment partially clogs the gravel/cloth filters or the outside of the perforated pipe itself, enough water can flow through the remaining holes to satisfy the design release.

Slotted Standpipe from Low Flow Orifice, Inlet Control (dry pond, shallow wet pond, or shallow marsh) [Figure 3.3a]

In this Baltimore DPW design, an "L" shaped PVC pipe is attached to the low flow orifice. An orifice plate is located within the PVC pipe which internally controls the release rate. Slots or perforations are all spaced vertically above the orifice plate, so that sediment deposited around the standpipe will not impede the supply of water to the orifice plate.

Negatively Sloped Pipe from Riser (wet ponds or shallow marshes) [Figure 3.3b]

This design was developed in Montgomery County, Maryland to allow for extended detention in wet ponds. The release rate is governed merely by the orifice of the pipe. The risk of clogging is largely eliminated by locating the opening of the pipe at least one foot below the water surface where it is well away from floatable debris. Also, the negative slope of the pipe reduces the chance that debris will be pulled into the opening by suction. As a final defense against clogging, the orifice can be protected by wire mesh.

Hooded Riser (wet ponds) [Figure 3.3c]

In this design, the extended detention orifice is located on the face of the riser near the top of the permanent pool elevation. The orifice is protected by wire mesh and a hood, which prevents floatable debris from clogging the orifice.

EFFECTIVENESS IN STORMWATER CONTROL

Peak Discharge Control

Extended detention ponds are effective in controlling post-development peak discharge rates to the desired pre-development levels for the design storm(s) specified. The optimum level of flood control is achieved when multiple design storms are controlled. Recent modeling analyses suggest that control of both the 2 and 10 year design storm may be sufficient to adequately control the entire spectrum of expected flood frequencies (Md WRA, 1983a). Extended detention ponds are also capable of managing smaller floods that contribute to channel erosion problems that occur more frequently than the annual or two year flood.

Groundwater Recharge

Groundwater recharge in extended detention ponds is limited to the runoff that infiltrates through the pond bottom during the relatively infrequent periods when the pond is inundated. The total volume of recharge is negligible in comparison to that provided by infiltration BMPs.

Volume Control

The post-development increase in the total runoff volume from a site is not significantly changed by extended detention ponds.

Downstream Effects

As with other detention ponds, the desired downstream reduction in peak discharge associated with the two year flood may not be achieved in watersheds if the ponds are randomly sited, due to the location and timing of individual releases (Schueler and Sullivan, 1983; APWA, 1981; NVPDC, 1979). For example, an extended detention pond near the bottom of a watershed may detain stormwater just long enough to coincide with the arrival of the upstream peak, and actually add to the peak discharge at that point. Therefore, it is advisable to perform detailed watershed modeling, such as TR-20 (SCS, 1982), to evaluate the cumulative hydrological impact of ponds on the total watershed hydrograph, and locate ponds and adjust release rates accordingly. Generally, watershed timing problems are not a concern during smaller, more frequent storms because of the low discharge rates of extended detention ponds.

Streambank Erosion Control

Peak-shaving detention ponds have traditionally been thought to reduce the extent of downstream channel erosion when geared to control the two year storm. This belief has its roots in the the early research of Wolman and Leopold which demonstrated that bankfull discharges, that occur on average every 1.5 to 2 years, control the shape and form of natural channels. Many local governments have subsequently adopted stormwater management policies that require the post-development peak discharge for the two year storm be controlled to pre-development levels. However, keeping the post-development two year design storm within the banks is normally not sufficient to prevent downstream bank erosion, since the two year flood is itself an erosive condition.

As noted by Andersen (1970) and Leopold et al (1964), streambank erosion can only be controlled when both the magnitude and frequency of the post-development two year flood are adequately managed. After a watershed is developed, small intense storms can dramatically increase the frequency with which two year bankfull discharges occur. The increased number of bankfull floods, in turn, increases the probability of downstream bank and channel erosion. The higher frequency of bankfull flooding after development can be readily demonstrated using the Rational formula:

(EQ 3.1) $Qp = (C)(i)(A)$

where Qp = instantaneous peak discharge (cfs).
C = runoff coefficient.
i = rainfall intensity for a duration equal to the watershed time of concentration (inches/hour).
A = watershed area (acres).

If Qp is set equal to the 2 year bankfull discharge, A is held constant, and the runoff coefficient is expressed as the ratio of pre- to post-development values, the equation can be rearranged to solve for the minimum rainfall intensity producing the two year flood as a function of watershed development:

(EQ 3.2) $Ia = i / (Cb/Ca)$

where Ia = minimum post-development rainfall intensity that produces bankfull flow.
i = rainfall intensity for the two year storm.
Ca = post-development runoff coefficient.
Cb = pre-development runoff coefficient.

Values for i as a function of C are shown in Table 3.1. As an example, an undeveloped watershed with a 15 minute time of concentration (Tc), will generally experience a two year flood after a 15 minute rainfall depth of about 0.9 inches. However, after the watershed becomes 50% impervious, only 0.2 inches of rain over a 15 minute period are needed to produce the same peak discharge rate.

Table 3.1: Limiting Rainfall Depths (inches) for Different Storm Durations that Produce Bankfull Flow Conditions

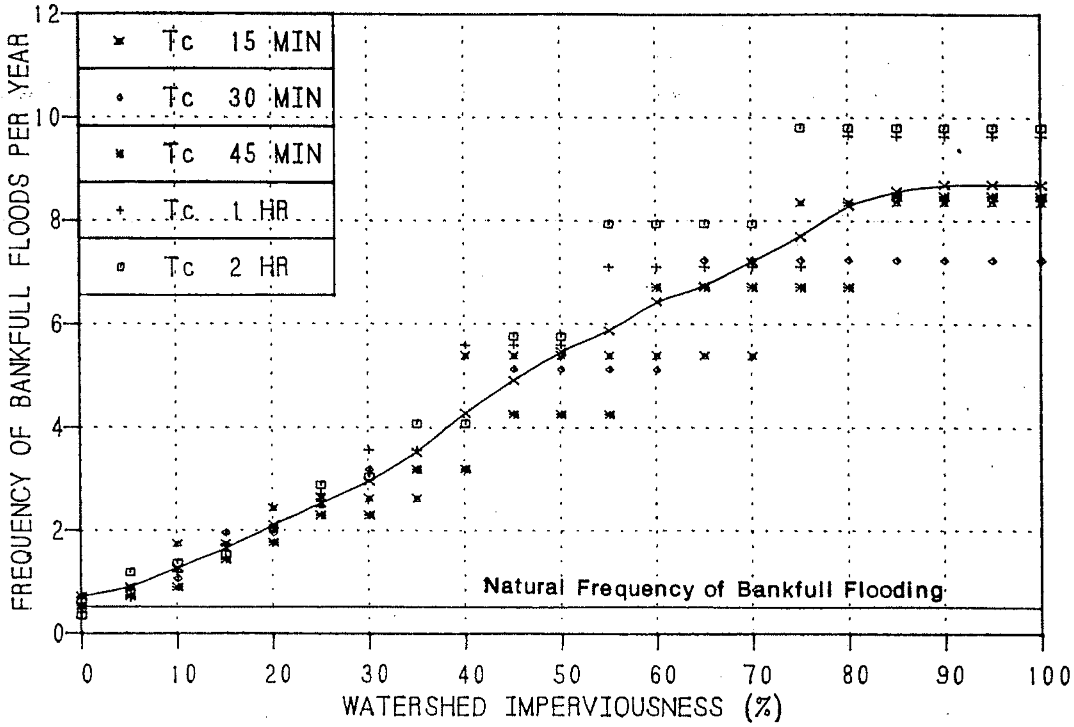
Rainfall Intensity for Watershed Time of Concentration (inches/interval)	Runoff Coefficients								
	.1	.2	.3	.4	.5	.6	.7	.8	.9
15 minutes	.88	.44	.30	.22	.18	.15	.13	.11	.10
30 minutes	1.20	.60	.40	.30	.24	.20	.17	.15	.14
45 minutes	1.33	.67	.45	.33	.27	.22	.19	.17	.15
1 hour	1.40	.70	.47	.35	.28	.23	.20	.18	.16
2 hours	1.80	.90	.60	.45	.36	.30	.26	.22	.20
3 hours	1.80	.90	.60	.45	.36	.30	.26	.22	.21
24 hours	3.10	1.55	1.03	.80	.62	.53	.43	.38	.34

Estimating Post-Development Bankfull Flooding Frequency

The number of bankfull floods expected to occur each year under various levels of development can be crudely calculated by solving the Rational formula for individual storms over a long period of time, and recording the frequency that postdevelopment peak discharges equal or exceed the pre-development bankfull discharge level.

Figure 3.4 was constructed in this manner, utilizing a six year record of maximum monthly short-term precipitation intensity values from the Greenbelt, Maryland station (NOAA, 1977-83). The bankfull flooding frequency analysis is described in greater detail in Appendix B. As can be seen in the figure, bankfull discharges normally occur once every two years in undeveloped watersheds (zero imperviousness). At a moderate level of development (25% imperviousness), bankfull floods are predicted to occur about twice a year, on average. Once a watershed becomes completely impervious, bankfull flows are expected to occur on nearly a monthly basis.

Figure 3.4: Increased Frequency of Bankfull Flow Conditions as a Result of Watershed Imperviousness, for Selected Values of Watershed Time of Concentration (Tc)



Effect of Extended Detention on Bankfull Flooding Frequency

The net effect of extended detention is to store a fraction of the incoming storm runoff volume and release it later so that it does not materially influence the uncontrolled postdevelopment storm hydrograph. Figure 3.5 shows the estimated effect of extended detention storage on the frequency of bankfull discharge under varying degrees of watershed development. Again, the assumptions and techniques underlying the analysis are described in detail in Appendix B. As shown, the curves suggest that extended detention storage equivalent to the runoff volume produced by a one-inch storm should be capable of reproducing the natural frequency of bankfull flooding, and thus reduce the probability of downstream erosion.

Local governments have developed a number of sizing rules for extended detention; each rule specifies both a volume to be detained and a duration over which this volume is released. Some of the extended detention sizing rules used in the region include:

SIZING RULE 1: A volume equivalent to one-half inch of runoff distributed over the contributing watershed released over a 40 hour period (Montgomery County, Maryland, DEP, 1984a).

SIZING RULE 2: The runoff volume generated from the one year, 24 hour storm be released over a minimum of 24 hours, equivalent to 2.6 inches. (Md WRA, 1985b; Prince Georges County, Maryland, DER, 1984).

SIZING RULE 3: The runoff volume generated from the two year, 24 hour design storm released over 24 hours (about 3.2 inches).

SIZING RULE 4: The runoff volume generated from a one inch storm released over 24 hours.

SIZING RULE 5: A volume equivalent to a land use dependent runoff depth distributed over the contributing watershed, released over a 40 hour period (Chart A: NVPDC, 1980; Fairfax County DEM, 1980).

SIZING RULE 6: The "first flush" runoff volume (i.e., one-half inch per impervious acre) released over 24 hours (Md WRA, 1986b).

Based on the bankfull flooding analysis presented in Appendix B, and shown in Figure 3.5, it is apparent that all of the sizing rules should be capable of reducing bankfull flooding episodes sharply. As might be expected, the sizing rules that specify the greatest detention volume (2 to 5) should be able to reduce bankfull flooding frequencies to at or below natural, pre-development levels.

As a final check, the extended detention release rates from the pond should be evaluated to determine if they are still erosive. A handy procedure (Helfrich, personal communication) is to find the nearest natural channel cross section below the pond, and calculate the discharge rate for the channel running a quarter full, assuming moderate but non-erosive stream velocities (3-5 feet per second). If the extended detention release rate exceeds the calculated discharge for the natural channel, the design detention time should be increased incrementally until the the release rate is smaller than the channel discharge. The procedure is illustrated in example 3-1.

EXAMPLE 3-1: CALCULATING NON-EROSIVE EXTENDED DETENTION RELEASE RATES

Step 1. Determine Average Discharge (cfs) for Extended Detention Release (Qex).

Qex = S/T

where S = detention storage volume (cubic feet).
T = detention time (hours*3600 secs/hour).

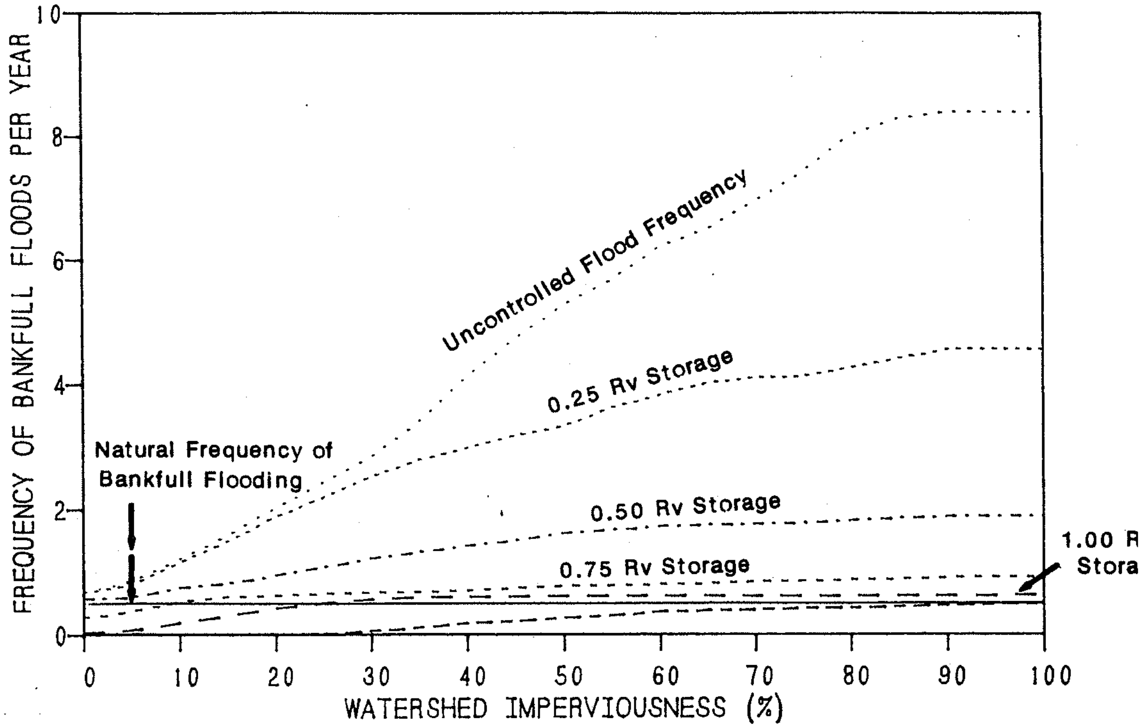
Step 2. Determine Discharge For Natural Channel Segment (Qc).

Qc = 0.25 (Xc)(Va)

where Xc = Channel Cross-sectional area (square feet).
Va = permissible velocity (3-5 feet per second).

Step 3. If Qex is greater than Qc, then repeat Steps 1 and 2, using either a longer detention time (T) or a smaller detention storage volume (S), until Qex becomes less than Qc.

Figure 3.5: Effect of Extended Detention Storage on Bankfull Flooding Frequency



NOTE: Extended detention storage (STOR) equals the volume equivalent to the given rainfall depth times the runoff coefficient.

POLLUTANT REMOVAL

Settling is the primary pollutant removal mechanism associated with extended detention. As such, the degree of removal is dependent on whether a given pollutant is in particulate or soluble form. Removal is likely to be quite high if a pollutant is particulate, whereas very limited removal can be expected for soluble pollutants. Unfortunately, some of the urban pollutants of greatest concern occur primarily in soluble forms (e.g, nitrate and ortho-phosphorus). Removal of these soluble pollutants may be obtained if the lower stage of the extended detention pond is managed as a shallow wetland to utilize natural biological removal processes.

Settling Behavior of Urban Pollutants

The settling behavior of urban pollutants has been evaluated in a series of laboratory and field studies. Grizzard et al. (1986), Driscoll (1986), and Whipple and Hunter (1981) have utilized experimental settling column data to assess pollutant settling behavior over time. In each study, urban runoff was introduced into four to six foot deep plexiglass chambers and the change in pollutant concentration over time was measured at sampling ports located at different depths on the column. In addition, the long term pollutant removal performance of two extended detention ponds have been evaluated in local field monitoring efforts. During the Washington NURP study (MWCOG, 1983b) a dry pond (Stedwick) in Montgomery County, Maryland was modified to achieve 6-12 hours of extended detention, and monitored over a 18 month period. Interim results are also available for an extended detention pond (London Commons) monitored in suburban Northern Virginia (OWML, 1986a). Together, these studies provide a basis for estimating the detention time needed to obtain maximum possible removal for specific pollutants of interest listed below.

SEDIMENT

The settling column experiments indicated that 60-70% of urban sediments settle out within the first six hours. The remaining sediment may take as much as 2 days to settle out (Figure 3.6). Maximum removal rates after 48 hours of detention ranged from 80-90%. The rather slow sediment settling rates are primarily due to the very fine-grained particle distribution of sediment in urban runoff (OWML, 1983). Washington NURP field monitoring at the Stedwick extended detention pond generally supports the lab measurements. The pond was estimated to remove 65% of incoming sediment over the long term (MWCOG, 1983a), which is similar to the 6-12 hour removal rate reported in the settling column study (Figure 3.7). An average storm removal of approximately 65% was also reported for the London Commons pond (OWML, 1986a), which also experienced relatively brief detention times (estimated at 6-12 hours).

PHOSPHORUS

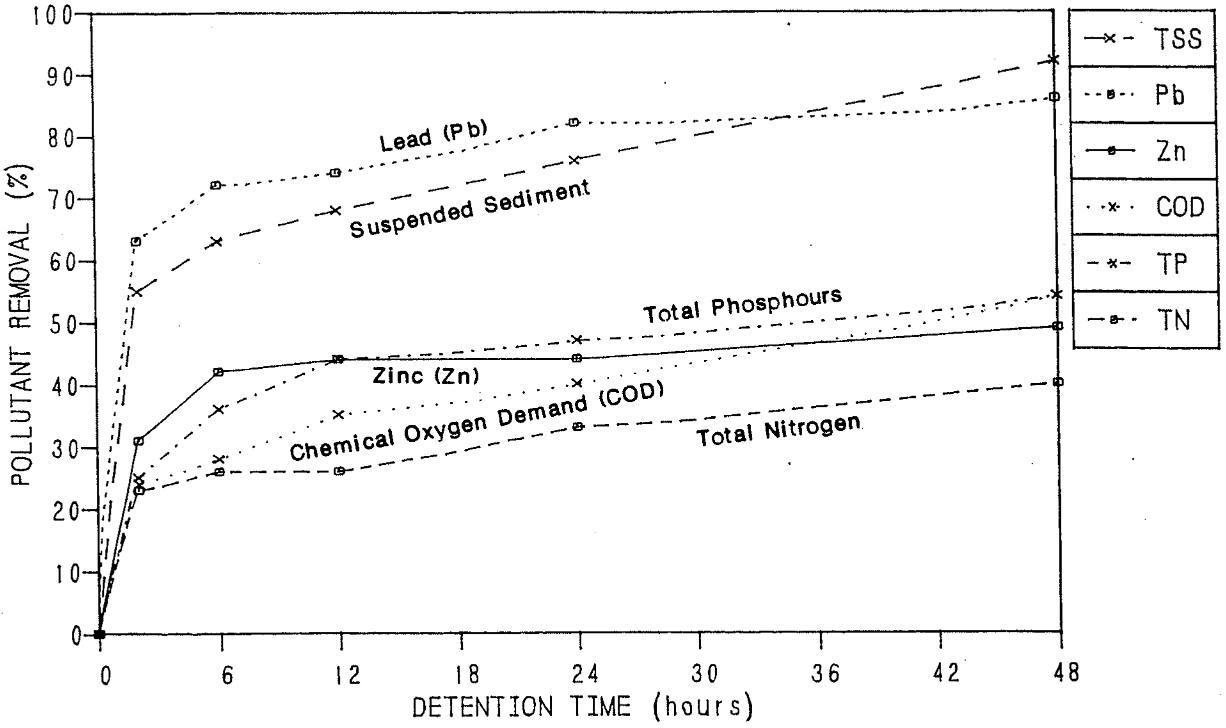
Both settling column studies indicated a maximum upper limit of about 40-50% removal for total phosphorus after 48 hours, with most of the removal occurring within the first 6 to 12 hours. The upper limit for phosphorus removal by settling is due to the fact that soluble forms comprise over half of all phosphorus found in urban runoff (Chapter 1). Nearly all the particulate phosphorus settled out in the OWML experiments, accounting for the majority of observed removal. In addition, a small fraction of soluble phosphorus adsorbed to sediment and eventually settled out during the experiments.

The field studies showed variable performance in removing phosphorus. Less than 15% of total phosphorus was removed at the Stedwick site over the long-term; whereas, initial results at the London Commons site indicated much higher average (70%) total phosphorus removal (OWML, 1986a). However, it is very likely that the long-term total phosphorus removal at the site is much lower, since very low (or even negative) removal rates were reported for larger storms. Resuspension of total phosphorus was cited as the likely cause.

NITROGEN

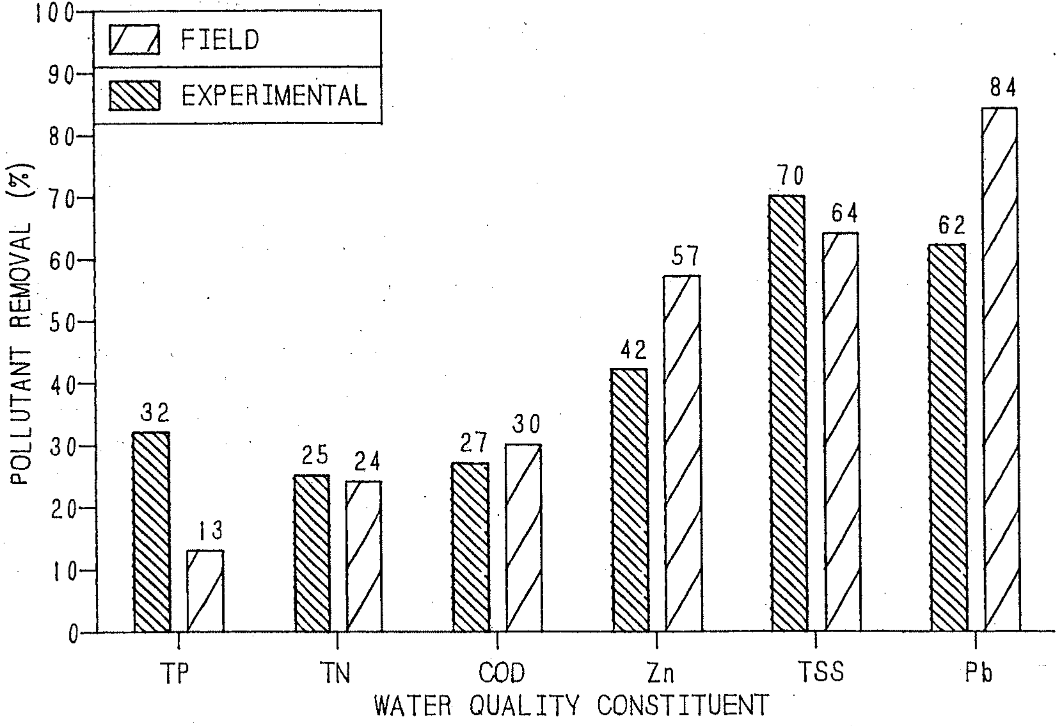
In the OWML (1983) settling column experiments, the upper limit on nitrogen removal achieved after 48 hours of detention was about 40%. Again, this is due to the predominance of soluble forms of nitrogen that comprise about 70-80% of the total nitrogen found in the Washington, D.C. area urban runoff (NVPDC, 1983). Field studies at the Stedwick extended detention pond suggested a long-term total nitrogen removal rate of about 25%, which compares well with the lab studies (Figure 3.7). Almost all of the particulate nitrogen settled out from the pond, but only limited settling of soluble nitrogen forms was reported. A higher average storm removal of total nitrogen was reported (52%) at the London Commons site (OWML, 1986a), although the long-term removal rates may not be as high.

Figure 3.6: Removal Rate vs. Detention Time For Selected Pollutants



NOTE: Based on OWML (1983) settling column data. Average values for seven tests. Removal equivalent to 4 feet of settling.

Figure 3.7: Urban Pollutant Removal After 6 to 12 Hours Detention Time Comparison of Lab Studies and Field Measurements



ORGANIC MATTER

Organic matter, as measured by BOD in Whipple and Hunter (1981) and COD in OWML (1983), exhibited similar settling behavior in the column tests. Average maximum removal after 32 and 48 hours, respectively, was about 40-50%. Organic matter exhibited rapid settling rates over the first 6-8 hours, followed by gradual but steady removal thereafter. Long-term COD removal rates at the Stedwick site were on the order of 30%, which compare favorably to the six-hour detention removals observed in the lab (Figure 3.7).

TRACE METALS

Settling of most trace metals in the column tests was initially quite rapid. Lead, which has a close affinity with suspended sediment, exhibited essentially similar settling behavior (Figure 3.6) (Whipple and Hunter, 1981). Maximum average removal after 48 hours was greater than 90%, with about two-thirds of the settlement occurring within the first six hours. Long-term lead removal measured in the field was even greater, with 84% removal recorded after the first 6 hours. Maximum removal of zinc was much lower, averaging about 50% in the OWML experiments and about 30% in Whipple's. Unlike lead, most of the zinc (<70%) in urban runoff is in soluble form (NVPDC, 1983). However, a significant portion of the soluble zinc appears to adsorb to sediment particles and settle out of the water column. This appeared to be the case at the Stedwick site, where long-term removal rates were estimated to be near 60%, despite the fact that less than 20% of the incoming zinc was in particulate form at the site.

OTHER POLLUTANTS

Whipple and Hunter (1981) noted an order of magnitude reduction in bacterial counts after 32 hours of detention. Also, about 60-70% removal of hydrocarbons was reported over the same interval.

Additional Removal by Biological Means

Biological removal of soluble pollutants can be achieved by creating artificial wetlands in the lower stage of a dry extended detention pond. Marsh plants, algae and bacteria that grow on the shallow, organic rich sediments can take up soluble forms of nutrients needed for their growth. Also, the marsh sediments are an excellent substrate for pollutant sorption. The degree of pollutant removal attained in shallow wetlands is uncertain, but appears to be dependent on the size of the wetland in relation to pollutant load delivered to it (Nichols, 1983). Removal varies seasonally, with the most removal during the growing season, and the least removal occurring in the late fall and winter after the plants have died back.

Wetlands can sometimes become a net source of nutrients in the fall and winter months, as nutrients stored in above-ground plant tissue are "pumped out" to the water column during senescence. Indeed, the only permanent sinks for pollutants in an artificial marsh are gradual burial in the sediments, harvesting, and occasional episodes of denitrification. However, even though much of the incoming nutrient load may only be temporarily stored in wetlands, the nutrients are released at a time of the year when they will have the least direct impact on receiving waters.

DESIGN TIPS FOR ENHANCING POLLUTANT REMOVAL

Detention Time

For water quality purposes, detention times of at least 24 hours are probably necessary to achieve maximum removal of most pollutants. While most of the settling occurs within the first 12 hours in the settling column experiments, it is advisable to provide further detention since several hours may be needed before ideal settling conditions develop in a pond. Slightly longer detention times may be needed for downstream channel erosion control if indicated by the method outlined in Example 3-1 (Helfrich, personal communication).

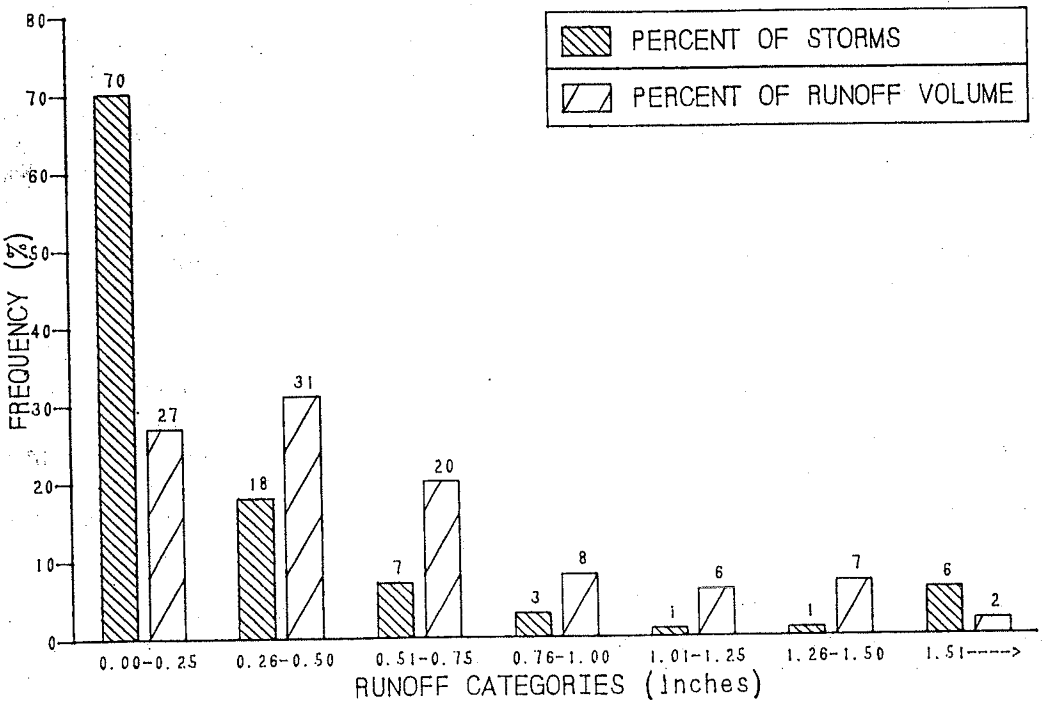
Achieving Adequate Detention For All Storms

One of the most difficult problems in extended detention design involves sizing the control device so that it provides adequate detention time for the entire spectrum of storms. For example, if an extended detention pond is designed to store and release the one-year storm over a 24 hour period, storms smaller than the one year storm event will pass through the orifice much more rapidly, and in some cases, may only have an average detention time of a few hours. Unfortunately, small storms deliver a majority of the annual runoff volume to the pond (Figure 3.8). As a result, the annual pollutant removal of the extended detention pond may be reduced if the small storms are not adequately detained.

Therefore, it is recommended that the pond designer perform several storage routing calculations (TR-20 method or equivalent) to determine the approximate detention time for the smaller, more frequent runoff events. Grizzard et al. (1986) suggest that as a target the average detention time in the pond should be 24 hours for the entire spectrum of storms each year. This can be done if the maximum detention time for the maximum detention volume is about 40 hours. Figure 3.8, which shows the approximate size distribution of storm runoff events in the Washington, D.C. area for moderately developed small watersheds, can be used to estimate inflow hydrographs for small storms for the routing calculations (i.e., the runoff volumes can be converted to SCS Triangular Unit Hydrographs using the methods outlined in Appendix B).

As a general rule, it is recommended that the average detention time for small runoff events (0.1-0.2 inches) should be no less than six hours.

Figure 3.8: Frequency Distribution Of Runoff Events in Moderately Developed Watersheds



NOTE: Based on Washington D.C. metropolitan area data;
300 storms, seven sites, I=10-30%.

Quantity Detained

The amount of runoff detained heavily influences the pollutant removal performance of an extended detention pond. Incoming runoff is only partially treated if a storm exceeds the detention storage volume provided in the pond. At a minimum, extended detention ponds should be sized to accommodate the runoff produced by the mean storm, and preferably should be capable of storing the runoff volume of a one-inch storm. However, in many cases, the stricter storage requirements recommended above for streambank erosion control (1.0-1.5 inches * Rv) will govern how much extra detention storage is needed.

Pond Shape: Two Stage Design

A two stage extended detention pond is recommended to improve pollutant removal and reduce maintenance requirements. Basically, the upper stage of the pond is intended to be dry except during large infrequent storms, whereas the lower stage is sized to accept regular inundation. As a general rule, the lower stage should have a minimum volume equivalent to:

(EQ 3.3) $Volb = [(Rm)(Rv)/12]\ (A)$

where Volb = volume of bottom stage (acre-feet).
Rm = volume of mean storm (0.4 to 0.5 inches).
Rv = rainfall/runoff coefficient (see Chapter 1).
A = area of contributing watershed (acres).

The lower stage volume will be the site of the bulk of the pollutant removal, as it will normally handle about 50-90% of storms in a given year (see Figure 3.8). Care must be taken to prevent the resuspension of previously deposited materials in the lower stage. This can be done by creating an artificial wetland to stabilize the bottom sediments, or by modifying the extended detention control to create a permanent pool. The risk of resuspending pollutants can be further minimized by installing a riprap apron or gabion baffle between the the pilot channel of the upper stage and the bottom of the lower stage. The two stage design (Figure 3.1) helps to reduce the velocity of runoff as it enters the lower stage, prevents concentrated flows from scouring or resuspending deposited sediments, and improves the overall settling characteristics of the lower stage.

Marsh Establishment

Wetland vegetation in the lower stage of an extended detention pond enhances removal of soluble nutrients and has several other benefits as well. Emergent marsh plants such as three-square, sedges, spatterdock, switchgrass and bulrush provide an attractive habitat for both wildlife and waterfowl, enhance sediment trapping, prevent sediment resuspension, and conceal trash and debris that normally accumulate near the riser.

Studies of the capacity of wetlands to assimilate wastewater indicate that they perform best when exposed to relatively dilute nutrient loads. Nichols (1983) presents summary data from many sites around the nation that suggests that maximum levels of nutrient removal can be achieved if loadings do not exceed 45 pounds of phosphorus or 225 pounds of nitrogen per surface wetland acre per year. Until more accurate criteria are developed from ongoing research on actual extended detention wetlands, these guidelines may be used to size artificial wetlands (see example 3-2).

Guidance on the methods for propagating and managing artificial wetlands is presented in detail in Chapter 9. The inlet-controlled slotted standpipe (Figure 3.3a). is probably the best control device for creating shallow wetlands in extended detention ponds because it can regulate water levels within the lower stage, and also maintain target detention times even when partially clogged.

EXAMPLE 3-2: DETERMINING THE FEASIBILITY OF USING A WETLAND TO AUGMENT EXTENDED DETENTION POLLUTANT REMOVAL

Given a 30 acre, 35% impervious townhouse development in a watershed that drains to an extended detention dry pond, calculate the volume of the lower stage of the pond and assess the feasibility of using the lower stage as an artificial wetland to augment pollutant removal:

STEP 1. The volume of the lower stage is equal to:
[(Rm)(Rv)/12](A) (Equation 3.3)
[(0.45)(0.36)/12](30) = 0.4 acre-feet.

STEP 2. The annual nutrient load to the lower stage is given by:
[(P)(Pj)(Rv)/12](C)(A)(2.72) (Equation 1.1)

for N = [(40)(0.9)(.36)/12](2.00)(2.72)(30) = 176 lbs/yr
for P = [(40)(0.9)(.36)/12](0.26)(2.72)(30) = 23 lbs/yr

STEP 3. Assume that the lower stage will be six inches deep to promote optimum wetland conditions. The area of the bottom stage is then: (0.4)/(0.5) = 0.8 acres

The average annual loading per wetland acre is:

(176)/(0.8)= 220 lbs/acre/yr of nitrogen
(23)/(0.8) = 29 lbs/acre/yr of phosphorus

STEP 4. Since the average annual wetland loading is below the recommended limits of 225 lbs/acre and 45 lbs/acre of N and P, respectively, the 0.8 acre artificial wetland should be large enough to provide significant pollutant removal.

Pilot Channels

Erosion will often occur within the low flow channel through the upper stage of an extended detention pond, unless it is stabilized by riprap. The lack of channel protection within the pond can actually make a pond a net sediment source (Schaefer, 1986; MWCOG, 1983b). However, pollutant removal is impaired if the pilot channel extends all the way through the lower stage to the riser, as sediment and other pollutants are often deposited on the pilot channel and can be subsequently resuspended. Optimally, in a two stage pond design, the stabilized low flow channel should extend to the lip of the lower stage of the pond.

Pond Slopes

The slopes leading to the pond should be gentle enough to prevent gully erosion of the banks during larger storms. Most local SWM guidelines suggest that side-slopes be no greater than 3:1 (h:v), and preferably flatter. Banks steeper than 2:1 (h:v) should be stabilized with riprap to prevent erosion. Gentle slopes make routine mowing of the banks easier and safer, allow easier pond access and are preferred by wildlife and waterfowl (Adams et al., 1983). The slope of the upper stage of an extended detention pond should be between 2 and 5% to promote rapid drainage.

Inlet and Outlet Protection

The stream channel immediately below the pond outlet should be lined with large stone riprap and graded to a slope of approximately 0.5% (MNCPPC, 1984) to prevent scouring during large storm events. A layer of filter cloth should be laid down that conforms to the natural dimensions of the channel, and then anchored with 18-30 inch stone riprap. Smaller sized riprap (9-12 inches) can be used if the diameter of the pipe outfall is less than 24 inches. Stilling basins can also be helpful in reducing the runoff velocity from the pond.

The invert elevation for inlet pipes should be as close to the surface of the upper stage as feasible. The outfall pipe should discharge at the bottom of the embankment directly to the outflow channel. Pipes that discharge above this level may cause erosion and undercutting of the embankment.

PHYSICAL SUITABILITY AT THE SITE LEVEL

Minimum Drainage Area

Dry extended detention ponds can theoretically be applied even on very small development sites (less than 10 acres). However, in practice, extended detention ponds are constrained by the minimum orifice diameter of the control device. Figure 3.9 shows the number of half-inch diameter holes needed to achieve desired detention times as a function of watershed size for each of the extended detention sizing rules outlined earlier (see EQ 3.4). As the example shows, less than ten half-inch diameter holes are needed in small watersheds under most of the sizing rules.

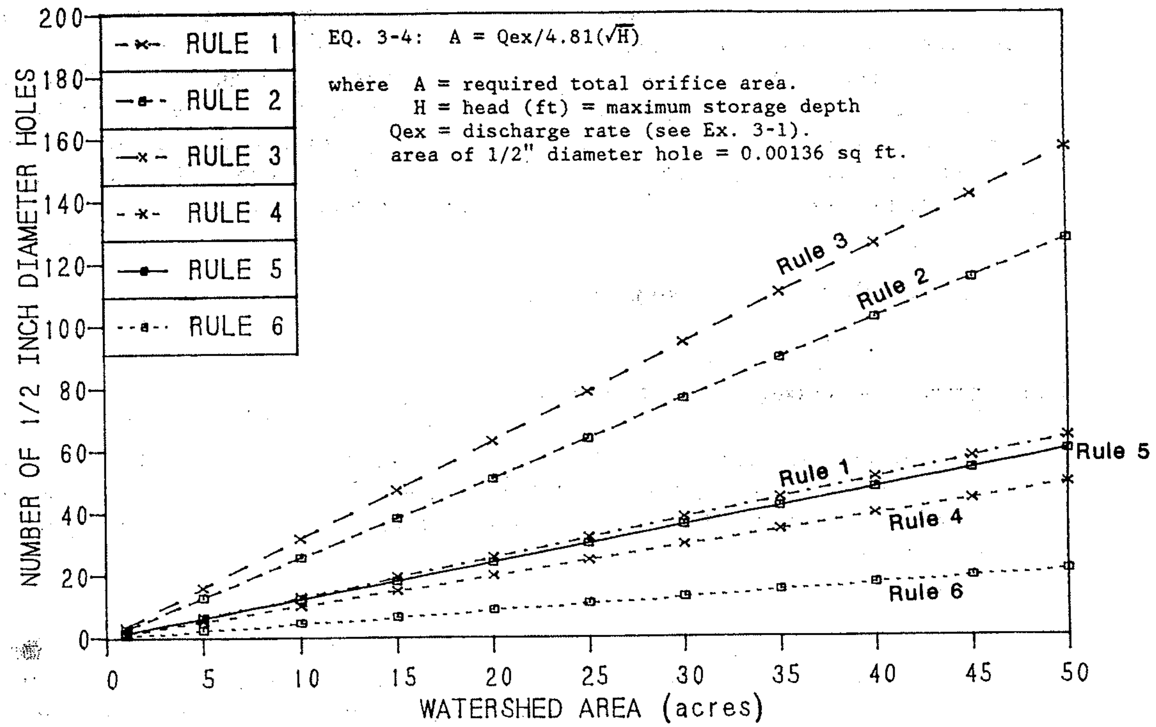
The minimum acceptable size for an orifice should be based on the engineer's confidence that it can remain unobstructed under realistic levels of future maintenance, given the kind of control device and the measures used to protect it against clogging. Under most current designs and maintenance programs, it is doubtful that control devices will function properly for long, if only a few half inch holes are used. As a result, several jurisdictions do not encourage dry extended detention on sites less tha n twenty acres in size.

Wet extended detention ponds often require larger drainage areas, as well. The larger area, however, is needed to maintain the permanent pool level rather than to prevent clogging (as wet extended detention ponds utilize submerged orifices that are not as susceptible to clogging (see Chapter 4).

Soils

While basin soils seldom prevent the application of extended detention, they should be checked when designing a pond. If soils are relatively impermeable (D soils), it is likely that a dry extended detention pond will experience problems with standing water. Conversely, if soils are very permeable (A soils) it will be difficult to establish an artificial wetland on the site.

Figure 3.9: Number of Extended Detention Orifice Holes (1/2 inch diameter) as a Function of Watershed Size. Assume H=3, and I=20%



Depth to Bedrock

If the bedrock layer lies close to the surface of the soil, it may become too difficult or expensive to excavate needed storage for an extended detention pond. Soil maps should be consulted, and soil borings need to be taken to confirm that no bedrock needs to be excavated.

Land Requirements

Extended detention ponds are not always feasible at sites where land costs or space are at a premium. This is particularly true when shallow artificial wetlands are used in combination with extended detention on small, heavily developed sites. In some cases, the shallow wetland pond and its buffer can consume as much as 10% of the site area. Normally, however, the space required for extended detention is less than 5% of the total site area.

Utility Relocation

Most utility companies will not allow existing underground pipes to be submerged under a permanent pool of water, as this can lead to infiltration/inflow problems and make maintenance efforts extremely difficult. Therefore, if wet extended detention with shallow marshes are to be used, the site designers should check to see if the pool area will cross any utility right of ways.

Wetland Permits

Often, the best place to put a pond on a site is in low marshy areas and natural depressions. Unfortunately, these areas are often classified as freshwater wetland habitat, and as such, may be protected under state or federal wetland laws. It is important to note that many wetland habitats are not easy to identify. The designer should consult local wetland maps and wetland permitting agencies to determine if the area has wetland status. If so, permits must be secured. See Herson et al. (1987) for a summary of local wetland requirements and the permitting process.

EXTENDED DETENTION POND COSTS

Predicting Extended Detention Pond Costs

A planning estimate of the base construction cost for a dry extended detention pond with greater than 10,000 cubic feet of storage can be approximated using the dry pond cost equation developed by MWCOG (Wiegand et al., 1986):

(EQ 3.5) $C = 10.71Vs^{0.69}$

where C = construction cost in 1985 dollars.
Vs = volume of storage (cubic feet) of the pond up to the crest of the emergency spillway.

As an example, if a dry extended detention pond is designed to have a total storage volume of 50,000 cubic feet (cf), the estimated construction cost for the BMP would be 10.7 (50,000)**0.69 or about $ 18,700. The actual cost will vary around this value, depending on the degree of excavation required. Costs can be significantly lower if natural depressions and topography are creatively used to reduce excavation requirements. The equation only estimates the labor and material costs involved in extended detention pond construction. It is recommended that about 25% should be added to estimate the extra "contingency" costs involved in designing the pond, securing the necessary permits, and overseeing construction. Therefore, the total construction cost of the dry extended detention pond in the example above would be 1.25 times $18,700 or approximately $23,400.

Generally, the extra storage needed for dry extended detention can easily fit within the much larger stormwater storage volume reserved in the pond for the 2 and 10 year storm, particularly if extra freeboard is available. An analysis of the incremental costs associated with modifying dry ponds for extended detention indicated that the extra storage volume and outlet modifications averaged about 10% more than the normal cost of constructing a existing dry pond (MWCOG, 1983a). The total incremental cost to modify a dry pond for extended detention seldom exceeds $2000 (Stack, 1987; MWCOG, 1983a).

Cost-Effectiveness

Extended detention ponds are the least cost urban BMP available that can both remove pollutants and control stormwater. While they are a cost-effective option for any sized development, economies of scale make dry extended detention ponds particularly attractive in moderate to large development areas (50 acres and above).

Creating an artificial wetland (or preserving a natural wetland) in the lower stage of a dry extended detention pond does not increase construction costs substantially. The total cost of grading the lower stage and purchasing/planting wetland stock should be less than $5,000 in most cases (Athanas, personal communication). Maintenance cost savings are realized when wetlands are created in the form of reduced mowing costs (i.e., a large and often soggy portion of the pond need not be mowed) and reduced length of costly pilot channels.

Over 3000 conventional dry ponds have been built throughout the Washington, D.C. metropolitan area since the early 1970s (MWCOG, 1985). Given the relative ease and low cost involved in modifying existing dry ponds to achieve extended detention, these ponds are ideal candidates for urban retrofit programs. Conversion of dry ponds presents a cost-effective opportunity to improve water quality in older, urbanized areas. Local governments, in some cases, may require a developer to convert an existing dry pond to compensate for not meeting stormwater quantity and/or quality requirements at a new development site, or alternately, fund a conversion program out of revenues collected from stormwater waiver fees.

MAINTENANCE REQUIREMENTS

Extended detention ponds have moderate to high maintenance requirements, depending on the extent to which future maintenance needs are anticipated during the design stage. Responsibilities for both routine and non-routine maintenance tasks need to be clearly understood and enforced. If regular maintenance and inspections are not undertaken, the pond will not achieve its intended purpose. For example, in two recent surveys, 40-50% of conventional dry ponds built in suburban Maryland were found to be structurally unsatisfactory as a result of poor or no maintenance (Geiss et al., 1984; Md WRA, 1986a). The basic elements of a dry extended detention pond maintenance program are described below.

Routine Maintenance

MOWING

The upper stage, side-slopes, embankment and emergency spillway of an extended detention dry pond must be mowed at least twice a year to discourage woody growth and control weeds. More frequent mowing may be required in residential areas by adjacent home-owners. This usually entails about 14 mowings annually, and constitutes the largest routine maintenance expense. Soggy conditions can make mowing costly and difficult within the pond unless a two-stage design is used. The use of native or introduced grasses which are water-tolerant, hardy and slow-growing are recommended. Some representative species, such as K-31 Tall Fescue, Crown Vetch, and Switchgrass are listed in the basin landscaping guide provided in Chapter 9 (see also Table 51 in Md SCS, 1983).

INSPECTIONS

Ponds should be inspected on an annual basis to ensure that the structure operates in the manner originally intended. When possible, inspections should be conducted during wet weather to determine if the pond is meeting the targeted detention times. In particular, the extended detention control device should be regularly inspected for evidence of clogging, or conversely, for too rapid a release. The upper stage pilot channel, and the flow path to the lower stage should be checked for erosion problems. Other problems which should be checked for include: subsidence, erosion, cracking or tree growth on the embankment; the condition of the emergency spillway; the accumulation of sediment around the riser; the adequacy of upstream/downstream channel erosion control measures; erosion of the pond's bed and banks; and modifications to the pond or its contributing watershed that may influence pond performance. Inspections should be carried out with as-built pond plans in hand.

DEBRIS AND LITTER REMOVAL

Debris and litter will accumulate near the extended detention control device and should be removed during regular mowing operations. Particular attention should be paid to floatable debris that can eventually clog the control device or riser.

EROSION CONTROL

The pond side-slopes, emergency spillway and embankment all may periodically suffer from slumping and erosion, although this should not occur often if the soils are properly compacted during construction. Regrading and revegetation may be required to correct the problems. Similarly, the riprap that connects the pilot channel of the upper stage with the lower stage may periodically need to be regrouted or repaired.

NUISANCE CONTROL

Standing water or soggy conditions within the lower stage of an extended detention pond can create nuisance conditions for nearby residents. Odors, mosquitos, weeds and litter are all occasionally perceived to be problems in dry ponds (Adams et al., 1983). Most of these problems are generally a sign that regular inspections and maintenance are not being performed (e.g., mowing, debris removal, clearing the extended detention control device). Nuisance problems can be concentrated into the lower stage if a two stage design is used. Also, wetland plants established in the lower stage can harbor birds and predacious insects that serve as a natural check on mosquitos, and will also conceal trash and debris.

Non-Routine Maintenance

STRUCTURAL REPAIRS AND REPLACEMENT

Eventually, the various inlet/outlet and riser works in a pond will deteriorate and must be replaced. Some local public works experts have estimated that corrugated metal pipe (CMP) has a useful life of about 25 years, whereas reinforced concrete barrels and risers may last from 50 to 75 years (MNCPPC, 1985). No stormwater management ponds have been in the ground for more than twenty years in the Washington region, and as a result, there is not much local experience in this area. However, since the various water works constitute about 25% of the initial construction cost (Wiegand et al., 1986), their replacement will be a significant future expense.

SEDIMENT REMOVAL

When properly designed, dry extended detention ponds will accumulate significant quantities of sediment over time. Sediment accumulation is a serious maintenance concern in dry extended detention ponds for several reasons. First, the sediment gradually reduces available stormwater management storage capacity within the pond. The best available estimate is that approximately 1% of the storage volume capacity associated with the two year design storm can be lost annually (a more precise estimate can be be made using the Simple Method in Chapter 1). Thus, as much as 20% of a pond's total storage capacity can be lost within 20 years. Even more storage capacity can be lost if the pond receives large sediment input during the construction phase. Second, unlike wet extended detention ponds (which have a permanent pool to conceal deposited sediments), sediment accumulation can make dry extended detention ponds very unsightly. Third, and perhaps most importantly, sediment tends to accumulate around the control device of dry extended detention ponds. Sediment deposition increases the risk that either the orifice or the filter medium will become clogged, and also gradually reduces storage capacity reserved for pollutant removal in the lower stage.

For these reasons, accumulated sediment may need to be removed from the lower stage every 5 to 10 years in a dry extended detention pond. More frequent spot clean-outs may be needed around the detention control device for some designs. Sediment removal operations are relatively simple if access for heavy equipment is provided. Front-end loaders or backhoes can be used to scrape off the bulk of the accumulated sediment, followed by manual removal of sediment deposited around the the control device. The disturbed area should be immediately stabilized with vegetation after removal operations are completed to prevent the control device from clogging again. The cost of mechanical sediment removal in extended detention ponds typically ranges from $5 to $10 per cubic yard (cy), depending on the size and accessibility of the pond. If an on-site disposal area is not available, then transport and landfill tipping fees may double or even triple the total cost of sediment removal operations.

The procedures and cost associated with sediment removal in wet extended detention ponds are somewhat different, and are discussed in greater detail in Chapter 4.

Although sediment removal must be performed more frequently in dry extended detention ponds than in wet ponds, the removal cost per clean-out cycle may be lower. One reason is that the sediment in extended detention ponds can dry out between storms, and consequently has a greater density than wet pond sediments (i.e., a ton of sediment will displace less volume in an extended detention pond). In addition, the relatively dry extended detention pond sediments do not need to be "de-watered" in special holding sites prior to disposal. Finally, the more expensive drag-line or hydraulic dredging methods required for sediment removal in larger wet ponds are not needed.

Total Maintenance Costs

The annual cost for routine maintenance in ponds averages about $300 to $500 per maintained acre (a "maintained acre" includes the pond and the surrounding buffer, and is generally equivalent to three times the surface area of the pond). Annual costs for non-routine maintenance (mainly sediment removal) are estimated to range from 1-2% of the pond's base construction cost. Therefore, it is recommended that homeowners and public works agencies budget 3-5% of the base construction cost of the extended detention pond, annually, to cover both routine and non-routine maintenance costs (Wiegand et al., 1986).

Design Tips to Reduce O&M Costs

1. The pond should have a two stage design with a top stage (2-5% grade) draining to a level, lower stage (Figure 3.1). Care should be taken during the grading phase to insure that no low pockets develop in the upper stage that might fill up with standing water. This design should make mowing operations easier on the top stage.

2. For easier mowing, sideslopes should be no steeper than 3:1 (h:v) and no flatter than 20:1 (h:v). Flatter slopes improve access and are generally safer, while steeper sloper help to prevent soggy conditions that impede mowing. Mowing costs can be sharply reduced if the pond buffer and upper stage are managed as a meadow rather than as a lawn. If this is done, the frequency of mowing can be reduced from approximately 14 to 2 operations a year (late spring and late fall).

3. All extended detention control devices should be surrounded by a filter of gravel or coarse stone and filter cloth. Externally regulated extended detention orifices are strongly recommended. All devices should have an accessible, above-ground cap to allow for easy clean-out.

4. Extra fill should be placed on the pond embankment to account for future settling or subsidence. An allowance of 10-15% is required in SCS pond designs (Md SCS, 1976).

5. Maintenance access must be provided to the pond by a public or private right-of-way, that has a minimum width of 10 feet and maximum slope of 15%. The maintenance access should never cross the emergency spillway, unless the spillway has been designed for that purpose and is properly stabilized. Lack of adequate access to ponds can lead to difficult and costly disputes over residential property damage during maintenance operations.

6. On-site disposal areas capable of receiving sediment from at least two clean-out cycles should be reserved in adjacent areas. The size of the required disposal area can be roughly calculated as shown in Example 3-3:

EXAMPLE 3-3: CALCULATING ON-SITE SEDIMENT DISPOSAL REQUIEMENTS

Step 1. Use the simple method (Chapter 1, Ex. 1-2) to determine the long-term sediment load to the pond.

Step 2. Based on the design detention time, determine the pond's sediment trapping capacity (from Figure 3.6).

Step 3. Compute the volume of trapped sediment; assuming one ton equals 0.8 cubic yards of periodically submerged sediment.

Step 4. Solve for area assuming the disposal area can accept a 24 inch depth of dry sediment per unit area.

7. Extra storage can be provided near the pond inlet or the lower stage to trap incoming sediments. This represents an extremely cost-effective means of reducing sediment removal costs, since removing a cubic yard of sediment after a dry extended detention pond is built is at least three

times more expensive than the cost of excavating it during construction (Wiegand et al., 1986). The minimum size for a sediment forebay located above the upper stage can be provisionally calculated using the first three steps in Example 3-3.

8. The responsibilities for both routine and non-routine maintenance need to be clearly vested so that funds can be budgeted for a regular maintenance program. If the responsibilities fall to a homeowners association, the nature and extent of their obligations should be clearly spelled out in a legally binding agreement or covenant. Even if a public agency is not responsible for maintenance, they should monitor and enforce private maintenance efforts as a normal part of the inspection process. Because of the limited financial reserves and technical expertise of homeowners associations, public maintenance is clearly preferable.

ENVIRONMENTAL ATTRIBUTES OF EXTENDED DETENTION

An extended detention pond is a significant modification to the urban landscape. The positive and negative impacts on both the natural and human environment should be carefully evaluated during the site review process.

Impacts to the Natural Environment

An extended detention dry pond can improve local wildlife habitat if an artificial wetland is created and/or the buffer is planted with plant species that provide food and cover for wildlife. As with most ponds, aquatic habitat in channels above the pond may be sacrificed since stormwater flows are not controlled. Unlike conventional wet ponds, however, dry and/or wet extended detention ponds can help to prevent degradation of downstream aquatic habitat by controlling channel destruction caused by post-development streambank erosion. Also, dry extended detention ponds do not generally release warm or anoxic water downstream due to their relatively brief detention times.

Impacts on the Human Environment

The primary impact of extended detention ponds on the human environment is related to aesthetic value. Most residents surveyed consider existing dry ponds to be fairly unattractive, unless they are maintained as a lawn (Geiss et al., 1984). Unlike wet ponds, most residents feel that dry ponds do not enhance property values, and in the case of poorly maintained ponds, can actually detract from them. While most residents do not consider dry ponds to be a safety hazard, many complain about mosquito and other nuisance problems (Adams et al., 1983). Both surveys indicated that dry ponds were perceived to have limited recreational, wildlife and aesthetic values, particularly in comparison to wet ponds and urban lakes.

Resident attitudes should be an important consideration in the planning and design of extended detention ponds, particularly when the same residents will eventually have to pay for maintenance. Careful attention should be paid to the landscaping value of the pond. Rectangular, steep-sided designs should be avoided. Where possible, natural depressions and lowlands should be utilized to make the pond as inconspicuous as possible. Moreover, since dry ponds will never have the amenity values of wet ponds, it makes sense to locate them as far from residences as possible.

Landscaping plans should be prepared for extended detention ponds. The plans should emphasize native plants and natural landscaping (i.e., using vegetation to break up any sharp angles created by the pond and pilot channel). A two-stage design for the pond is desirable to make the upper stage of the pond more suitable for regular mowing, or as a site for a wet meadow. The lower stage can then be managed as a wetland area. It is much more likely that citizens will accept a planned marsh in their community than if an unintended swampy area develops on its own. As with many BMPs, residents may need to be educated about the environmental benefits provided by the extended detention pond.

RELEVANT DESIGN GUIDANCE

The design summary presented on the following pages summarizes some of the more important design features to consider when planning an extended detention pond. The design features are also shown in schematic form in Figure 3.10. In addition, the following references should be consulted during final design:

Maryland Soil Conservation Service, 1981. Standards and Specifications for Ponds. Practice Code No. 378.

Maryland Association of Soil Conservation Districts, 1976. Stormwater Management Pond Design and Construction Manual.

DESIGN SUMMARY: EXTENDED DETENTION PONDS

- **QUANTITY DETAINED:**
  At a minimum, the volume of runoff detained should be equivalent to the runoff volume produced by a one inch storm. This volume is sufficient to achieve both high levels of particulate removal and downstream channel erosion protection for most of the storms that occur during a year. Higher levels of control can be achieved when the runoff volume from the one or two year storm is detained.

- **DURATION:**
  24 hours of extra detention are needed for optimal pollutant removal for the design detention volume. The control device should be adjusted so that smaller runoff events (0.1 to 0.2 inches), which normally pass through the pond quickly, are detained for at least a minimum of six hours. In larger watersheds, up to 40 hours of extended detention may be needed for streambank erosion control. As a final check, the runoff velocity of the downstream channel at the extended detention release rate should be computed to make sure that it is not erosive (Example 3-1).

- **TWO STAGE DESIGN:**
  A two stage pond design is recommended when extended detention is applied to dry ponds. The upper stage of the pond is sized and graded (2% minimum) to remain dry except during infrequent large storms, while the bottom stage is expected to be regularly inundated. The volume of the bottom stage should be set to store the runoff produced by the mean storm (approximately 0.45 inches; see EQ 3.3). The bottom stage will frequently be too wet to mow, and is best managed as a wetland or as a shallow pool. Both techniques act to prevent resuspension of previously deposited materials. Extra storage, over and above stormwater and extended detention requirements, should be provided within the bottom stage, or at the inlet to account for 20 years of sediment deposition.

- **WETLAND CREATION:**
  Wherever possible, a wetland marsh should be created in the bottom stage of an extended detention pond to help remove soluble pollutants that cannot be removed by conventional settling. Wetlands also provide wildlife habitat and hide unsightly debris and sediment deposits that frequently accumulate near the riser. The area of the wetland should be adjusted so that the average annual watershed loading (as computed by the Simple Method) does not exceed 45 pounds of phosphorus or 225 pounds of nitrogen per surface acre of wetland. Water depths of 6-12 inches are needed for optimal wetland growth. The wetland should be planted with native species which are suited to that environment (see Chapter 9 for further guidance on wetland plantings).

- **EXTENDED DETENTION CONTROL DEVICE:**
  In dry ponds, a vertical, internally controlled extension of the low flow orifice is the most trouble-free design, since it can withstand partial clogging and gradual sediment accumulation, and also can be used to set water levels. If the control device is below the ground surface, it should be protected with filter cloth and/or wire mesh, and encased in a trench of stone or gravel with a diameter greater than the orifice. The device should also have an above-ground extension, with a tight-fitting replaceable cap to facilitate clean-out. In wet ponds, a

negatively-sloped pipe protected by a wire mesh that extends from riser and withdraws water from at least a foot below the surface should be used.

- PILOT CHANNELS
  A riprap, concrete or paved low flow channel is required to route water through the upper stage of the extended detention pond. The pilot channel should end at the lip of the lower stage, where riprap or gabion baffles are placed to reduce velocities and spread out the flow path of the runoff reaching the lower stage, thus preventing scour and resuspension.

- SIDE-SLOPES:
  Side slopes should be no steeper than 3:1 (h:v), and no flatter than 20:1 (h:v).

- POND BUFFER:
  A minimum 25 foot wide buffer strip away from pond to the nearest lot should be reserved, and landscaped using low-maintenance grasses, shrubs and trees. A landscaping plan should be prepared for the pond and buffer, that improves the appearance for adjacent residents, meets specific design functions, and provides local wildlife habitat (see Chapter 9 for further details).

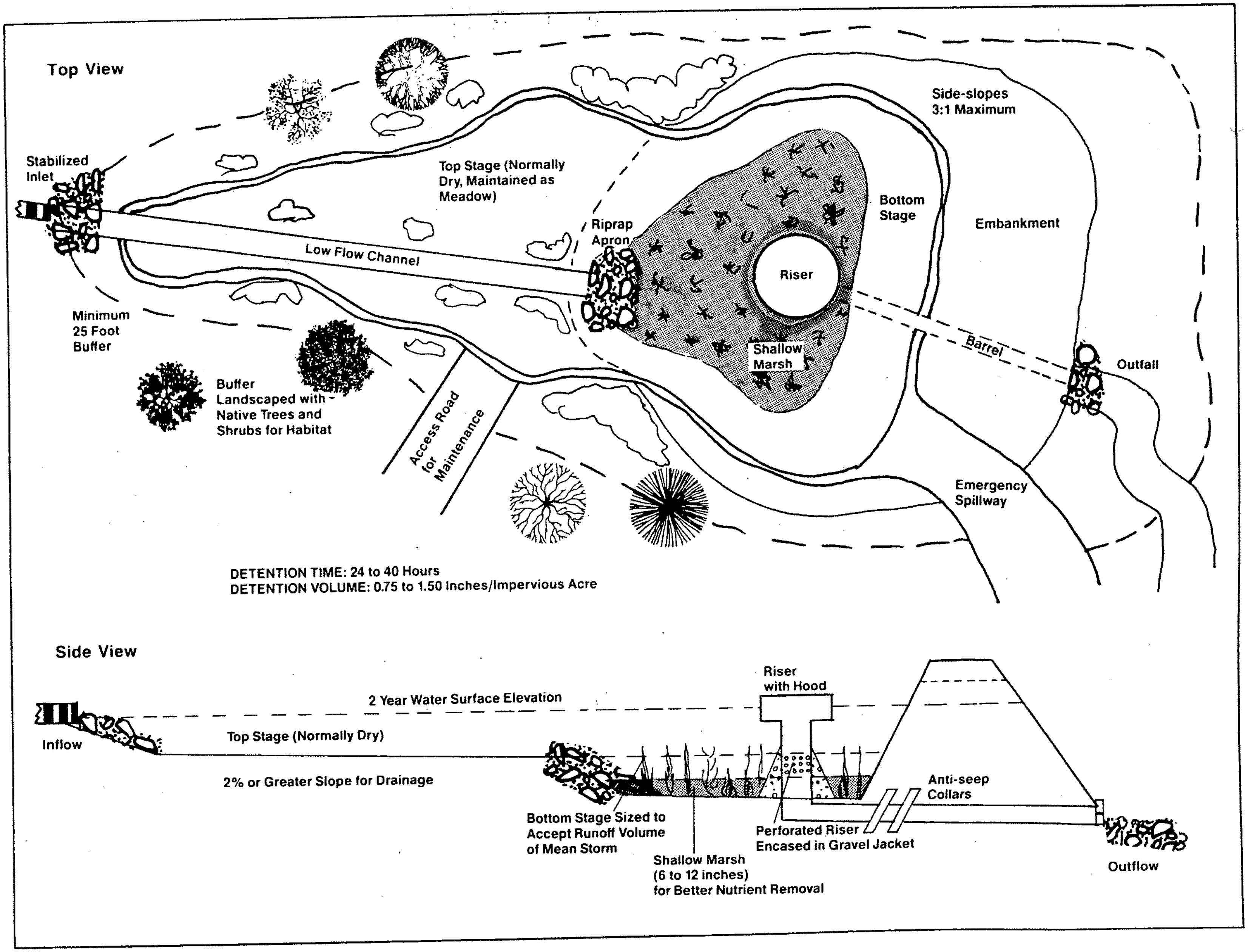
- EMBANKMENT:
  At least 10-15% extra fill should be allowed on the embankment to account for possible subsidence. The embankment should have at least one foot of freeboard above the emergency spillway. Anti-seep collars should be used to prevent seepage around the barrel. The embankment should be graded to allow access for heavy equipment, and should be mowed twice a year to prevent woody growth.

- SITE ACCESS:
  Adequate access from public or private right of way to the pond should be reserved. The access should be at least 10 feet wide, on a slope of 5:1 (h:v) or less, and stabilized to withstand the passage of heavy equipment.

- MAINTENANCE:
  Wet-weather inspections should be conducted annually, with as-built plans in hand. Inspections should emphasize the condition of the extended detention control device and low flow pilot channel. Extended detention facilities should be maintained as a meadow to reduce mowing frequency (2 times per year) and maintenance costs. Maintenance responsibilities should be clearly vested with funds reserved for both routine and non-routine activities.

- SEDIMENT REMOVAL:
  A five to ten year sediment clean-out cycle is recommended. Extra storage in the lower stage of the pond can be provided to accommodate sediment deposition. Also, on-site sediment disposal areas should be reserved to reduce removal costs. Do not begin final pond construction until upland area is stabilized.

Figure 3.10: Schematic of Extended Detention Pond Design Features

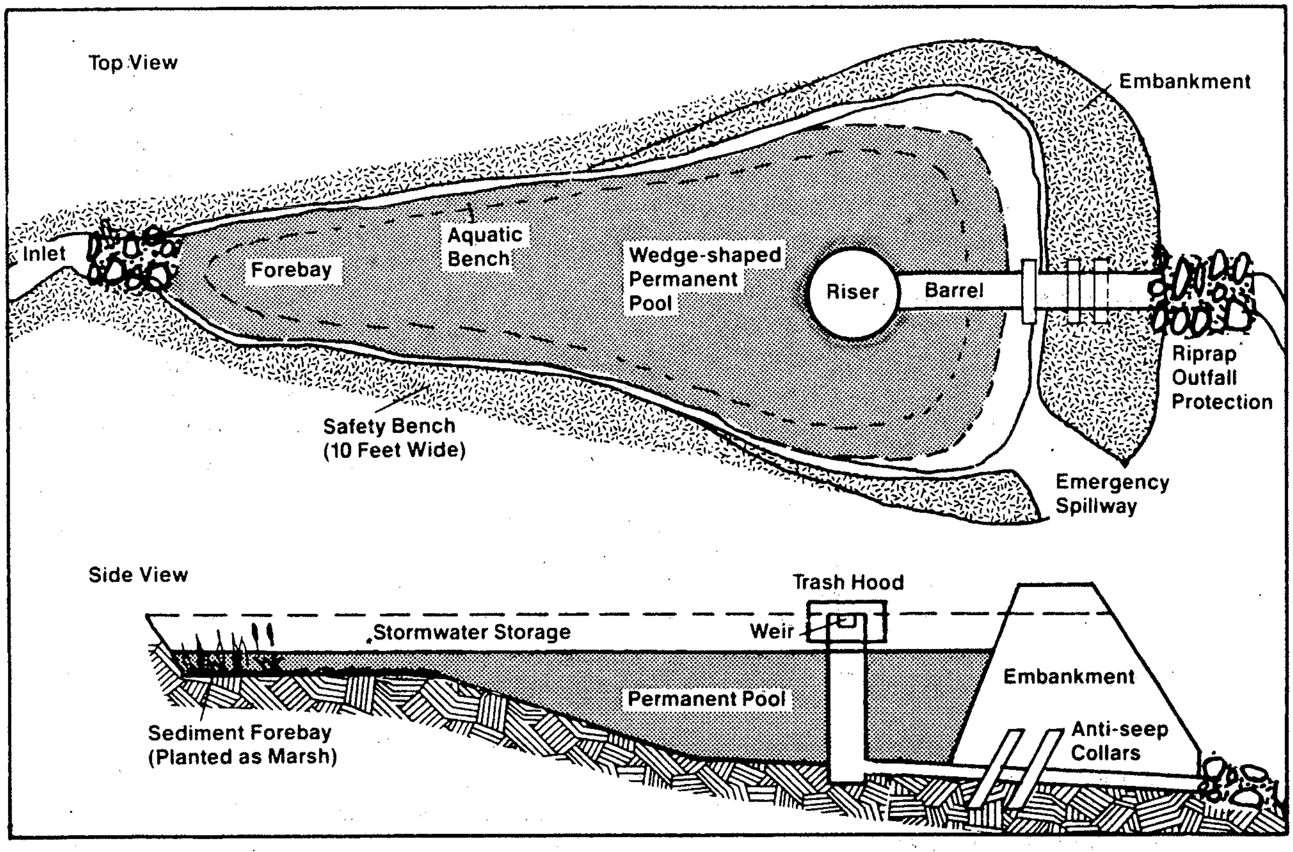


CHAPTER 4: WET PONDS

Wet ponds, also known as retention ponds, are an extremely effective water quality BMP. If properly sized and maintained, wet ponds can achieve a high removal rate of sediment, BOD, organic nutrients and trace metals. Biological processes within the pond also remove soluble nutrients (nitrate and ortho-phosphorus) that contribute to nutrient enrichment (eutrophication). Wet ponds are most cost-effective in larger, more intensively developed sites. Positive impacts of wet ponds include: creation of local wildlife habitat, higher property values, recreation, and landscape amenities. Negative impacts include: possible upstream and downstream habitat degradation, potential safety hazards, occasional nuisance problems (e.g., odor, algae, and debris), and the eventual need for costly sediment removal.

Perhaps more than any other BMP, wet ponds require careful planning and thoughtful design, and regular maintenance. Competing objectives must be reconciled at every potential pond site, and the final design may never achieve all of them. However, wet ponds are unique in that they can truly be a multi-purpose BMP, by providing stormwater management, pollutant removal and landscaping/habitat improvement.

Figure 4.1: Schematic of a Wet Pond



The best situation for employing wet ponds is in residential or commercial developments greater than twenty acres in size with a reliable source of water. Wet ponds can be an attractive feature in well-planned residential communities, particularly if a stable homeowners association exists to insure regular and non-routine maintenance. Otherwise, a public sector commitment to maintenance will be required.

EFFECTIVENESS IN STORMWATER CONTROL

Peak Discharge Control

Wet ponds can be effective in controlling post-development peak discharge rates to pre-development levels for desired design storms. Numerous methods exist for accurately defining stormwater storage needs by reservoir routing (SCS TR-20 and SWMM models, for example). As with any other peak-shaving facility, the optimal level of flood control is achieved when multiple design storms are controlled. Recent analyses suggest that control of both the 2 and 10 year design storm is sufficient to adequately control the entire spectrum of expected flood frequencies (Md WRA, 1983a).

Groundwater Recharge

Groundwater recharge in wet ponds is limited to the storage lost to infiltration through the pond bottom. The quantity of recharge is greater than that achieved in dry or extended detention ponds, but is negligible in comparison to infiltration basins and other volume control BMPs. In some sites, recharge may actually need to be to prevented to maintain a permanent pool.

Volume Control

The post-development increase in the total runoff volume from a site is not effectively modified by wet ponds. Some temporary control of runoff volume may be achieved when extra dead-storage is created by evaporation or infiltration. However, the volume control generally only occurs during minor storms in the summer months and after prolonged droughts.

Downstream Effects

As with all peak-shaving facilities, the desired downstream reduction in peak discharge may not be achieved in watersheds with many ponds because of the location and timing of individual releases (Schueler and Sullivan, 1983; APWA, 1981; NVPDC, 1979). As an example, a pond situated at the bottom of a watershed may detain stormwater just long enough to coincide with the arrival of the upstream peak, and actually increase the peak discharge. Therefore, it is advisable to perform detailed watershed modeling (such as TR-20) to evaluate the cumulative hydrological impact of wet ponds on the total watershed hydrograph, and locate ponds and adjust release rates accordingly.

Streambank Erosion Control

Peak-shaving wet ponds have traditionally been thought to reduce the extent of downstream channel erosion when high return storms (two year return frequency or less) are controlled. This belief has its roots in the early research of Wolman and Schick (1967) and Leopold (1968) which, demonstrated that bankfull discharges, which occur, on average, every 1.5 to 2 years, control the form of natural channels. Many local governments have subsequently adopted stormwater management policies which require the peak discharge of the two year storm be controlled to pre-development levels. However, after a watershed becomes developed, the frequency of bankfull discharges can increase markedly. In some highly impervious areas, bankfull discharges may occur as often as six times a year following development (Leopold, 1968 and Figure 3.4). The relatively small and frequent storms may have to be controlled to adequately protect streambanks from further erosion. This can be accomplished by extending the detention time of runoff within the wet pond by 24 to 40 hours. Suggested designs and sizing methods for incorporating extended detention into wet ponds are provided in Chapter 3.

POLLUTANT REMOVAL

The capability of wet ponds to remove pollutants borne in urban runoff has been demonstrated in local and national field studies (US EPA, 1983; MWCOG, 1983b). These studies have found pollutant removal to be variable from storm to storm, but generally high over the long-term, for well designed and maintained ponds. The degree of pollutant removal achieved by a pond is a function of the size and design of the permanent pool and the characteristics of individual urban pollutants.

Pollutant Removal Mechanisms in Wet Ponds

SEDIMENTATION

In theory, the incoming storm runoff displaces "old water" out of the pond and is then stored until the next storm. Suspended pollutants settle out from the water column to the pond sediments. Moreover, the permanent pool acts as a barrier to resuspension of deposited materials, improving removal performance over that achieved by dry ponds. The greatest initial settling often occurs near the inlet of the pond, where the velocity of the incoming runoff is dissipated by the still waters of the permanent pool. Settling in ponds during quiescent conditions can be modeled assuming Stokes Law Type I Sedimentation. Coarser materials are deposited first, followed by progressively finer-sized fractions. In practice, sedimentation is an effective pollutant removal mechanism unless short-circuiting occurs (i.e., incoming runoff passes through the pond without displacing the old water), or the volume of incoming runoff is greater than the volume of the permanent pool (in which case some portion of the runoff passes through the pond unmodified). As a result of these factors, pollutant removal rates often decline during larger storms in smaller ponds.

BIOLOGICAL UPTAKE

A unique feature of wet ponds is the presence of aquatic plants and algae that can remove significant amounts of soluble nutrients from the water column. Since soluble nutrients have minimal settling velocities, biological uptake represents an important removal pathway. In short, the plants convert the soluble nutrients into biomass which in turn can settle to the pond sediments. Once nutrients and organic materials are trapped in the sediments, they may be consumed by bacteria and removed from the system.

Estimates of Wet Pond Removal Efficiency

The pollutant removal capability of two wet pond facilities were evaluated during the Washington, D.C. area NURP study (MWCOG, 1983b; OWML, 1983). The wet ponds were found to be effective in removing particulate pollutants, with long-term average removal for the two ponds of 54% for sediment, 30% for chemical oxygen demand, 51% for zinc, 65% for lead, and approximately 20% for both organic nitrogen and phosphorus. In general, the removal of particulate pollutants in the wet ponds was very similar to that observed in extended detention ponds. Removal of organic materials was slightly lower in wet ponds in comparison to extended detention ponds, perhaps as a result of export of biomass and/or detritus from the ponds. The wet ponds were more effective in removing soluble nutrients with long-term removal of 60% of the nitrate and over 80% of the soluble phosphorus recorded during the course of the study. Uptake by algae and aquatic plants was apparently responsible for the removal.

Wet ponds monitored at other NURP projects (Tri-County RPC, 1983; US EPA, 1983; Driscoll, 1983b) followed the same pattern of pollutant removal observed in the Washington, D.C. area, with high sediment and trace metal removal, moderate removal of organic nutrients and COD, and apparently high removal of soluble nutrients. The absolute level of pollutant removal was found to be primarily a function of the ratio of pond volume to watershed size (US EPA, 1986; Driscoll, 1983b). Relatively undersized wet ponds had low and occasionally negative removal efficiencies, while moderate to large-sized ponds had correspondingly higher removal rates.

DESIGN TIPS FOR ENHANCING POLLUTANT REMOVAL

Pool Volume

The size of the permanent pool in relation to the contributing watershed is perhaps the single greatest factor influencing pollutant removal in wet ponds. Larger ponds remove pollutants better than smaller ones, and in general, "bigger is better". However, after a certain threshold size is reached, further removal by sedimentation is negligible. Also, an upper limit on pond size may be imposed by construction costs and site constraints. A number of wet pond sizing rules have been proposed to optimize pollutant removal. These alternative rules variously specify that the minimum volume of the permanent pool be equivalent to:

SIZING RULE 1:	One-half inch of runoff distributed over the contributing watershed area (Montgomery County DEP, 1984a).
SIZING RULE 2:	One-half inch of runoff distributed over the impervious portion of the contributing watershed (Md WRA, 1986b).
SIZING RULE 3:	Volume of permanent pool equivalent to a variable depth of runoff distributed over the contributing watershed, depending on land use (NVPDC, 1980; Fairfax County DEM, 1980).

SIZING
RULE 4: Two and a half times the volume of runoff generated from the mean storm over the watershed area (Md WRA, 1986c).

SIZING
RULE 5: Adjusted to achieve an average of two weeks of retention within the pond (Hartigan, 1986), or about 4 times the volume of runoff generated by the mean storm over the watershed area.

The first four pond sizing rules presume Type I sedimentation is the primary pollutant removal mechanism in wet ponds, while the fifth is designed to maximize biological uptake within the pond. The comparative impacts of each pond sizing rule on permanent pool volume, estimated pollutant removal, and construction cost were evaluated. Hypothetical land development scenarios described in Schueler et al. (1985) were used to determine the required permanent pool volume under each rule. The wet pond performance model of Driscoll (1983b) was adapted to project the estimated pollutant removal under each sizing rule (Figure 4.2). The output from the Driscoll model is somewhat conservative since it only considers removal by sedimentation during dynamic and quiescent (i.e., continuous and plug flow) conditions. However, it is very useful for comparing sizing rules, because; 1) pollutant removal is assumed to be a function of the ratio VB/VR (volume of the basin to the volume runoff generated from the mean storm) which is easily determined for each sizing rule, 2) the model has been calibrated to field data from numerous NURP pond monitoring sites, and 3) it generates long-term average removal rates from a statistical description of regional climate data.

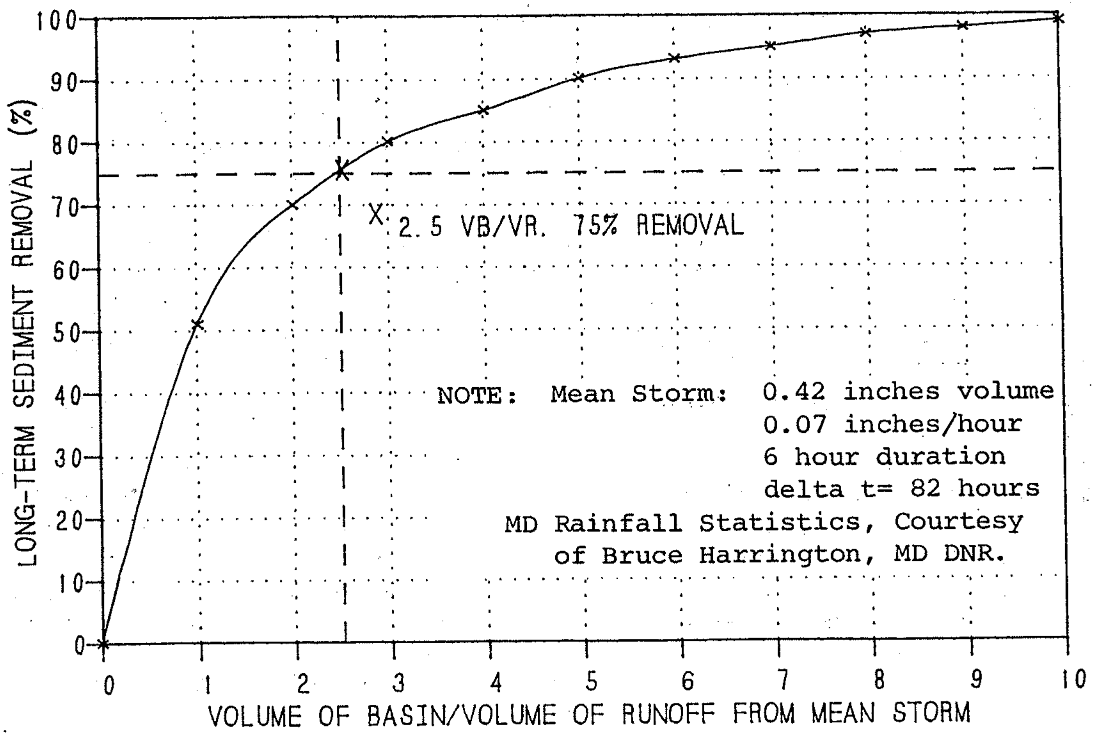
The different pond sizing rules create a great deal of variability in permanent pool storage volume for a given development scenario (Figure 4.3). Rules 4 and 5 create the largest permanent pool; rules 2 and 3 result in the smallest pools.

The impact of each sizing rule on the VB/VR ratio is shown in Figure 4.4. With the exception of Rule 1, all the sizing criteria have a constant or slightly decreasing VB/VR ratio as development becomes more intensive. VB/VR ratios for all of the pond sizing rules achieve or exceed an estimated long-term sediment removal of 60% (Figures 4.2 and 4.6). The larger ponds (VB/VR of 2.5 or more) are expected to achieve 71% sediment removal. Thereafter, additional small increments of sediment removal are only achieved by large increases in pool volume (Figure 4.2). Other urban pollutants have less rapid settling rates, and their long-term removal rates are proportionally lower than sediment (Figure 4.6).

The economic impacts of the sizing rules are depicted in Figure 4.5, in which the cost of constructing each hypothetical pond is compared with the cost of building a dry pond (the cheapest SWM alternative). The cost difference among the alternative rules is wide, with rule 5 producing wet ponds as much as 100% more expensive than dry ponds. Rules 2 and 3 are more economical with incremental costs of 25% or less.

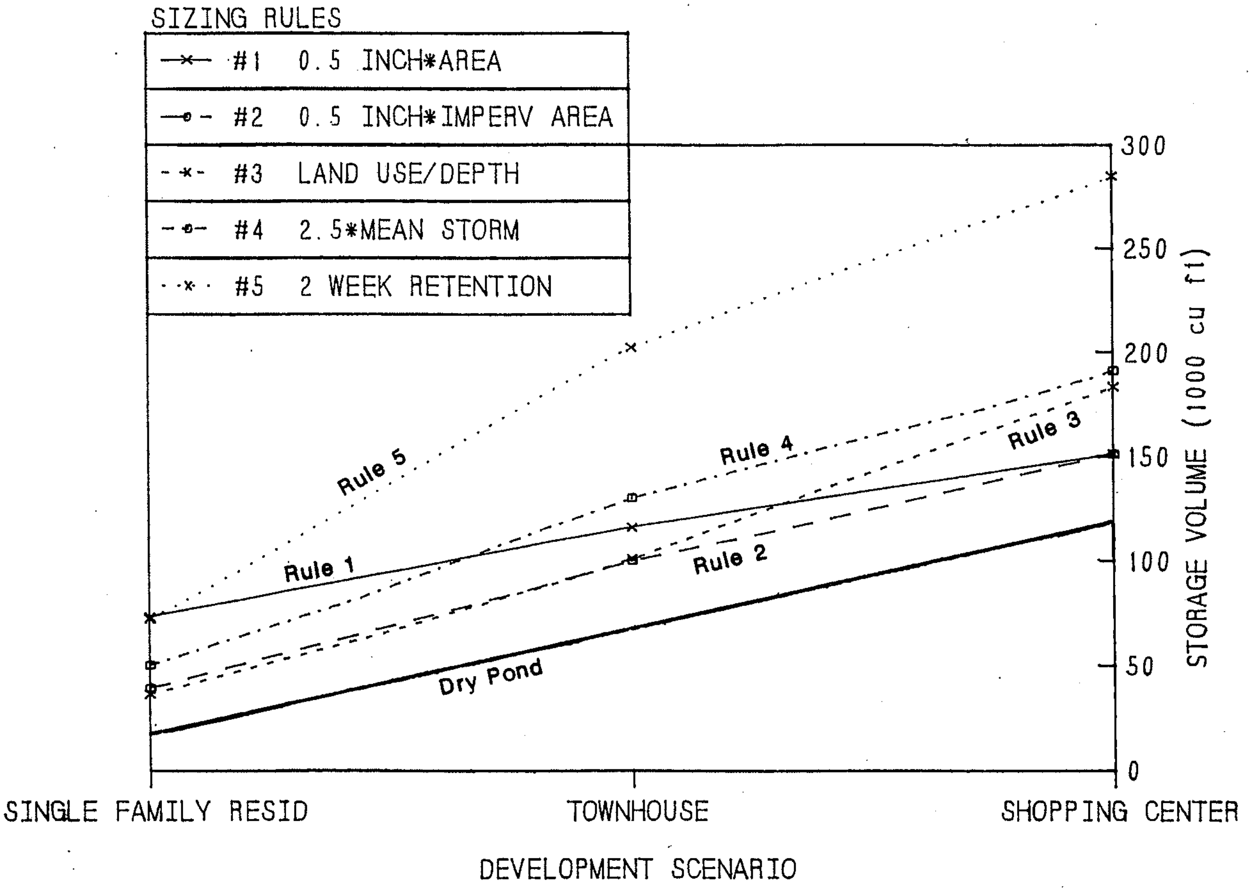
The choice of an appropriate pond sizing rule necessarily invites a trade-off between the degree of removal efficiency desired and the cost of achieving it. The comparative effects of each pool sizing rule are detailed in Table 4.1. Since all of the pool sizing rules produce at least moderate levels of sediment removal (60 to 90%), no individual rule is recommended here. The matter is properly the concern of local stormwater management policymakers.

Figure 4.2: Relationship Between VB/VR and Pond Sediment Removal



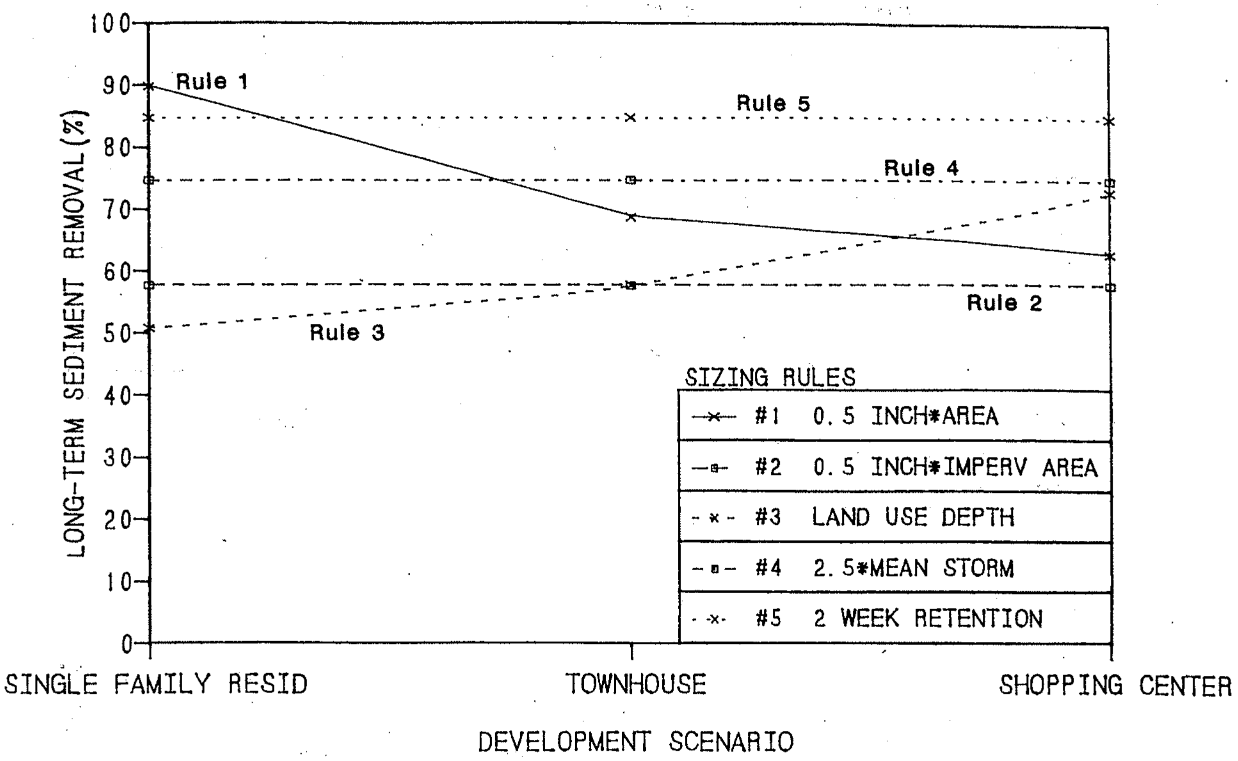
NOTE: Mean Storm: 0.42 inches volume
0.07 inches/hour
6 hour duration
delta t= 82 hours
MD Rainfall Statistics, Courtesy
of Bruce Harrington, MD DNR.

Figure 4.3: Effect of Pool Sizing Rules on Wet Pond Storage Volumes



NOTE: Storage includes 2 year design storm control.
Plot assumes a 25 acre watershed.

Figure 4.4: Effect of Pool Sizing Rules on Sediment Removal



NOTE: Plot based on VB/VR ratio, Driscoll (1983).
Plot assumes a 25 acre watershed.

Figure 4.5: Effect of Pool Sizing Rules on Construction Cost Increase Over Conventional Dry Pond, 2 Year Design

NOTE: Projected construction cost for wet ponds shown as the incremental cost over that for a 2 year design stormwater management pond. Assumes 25 acre watershed.

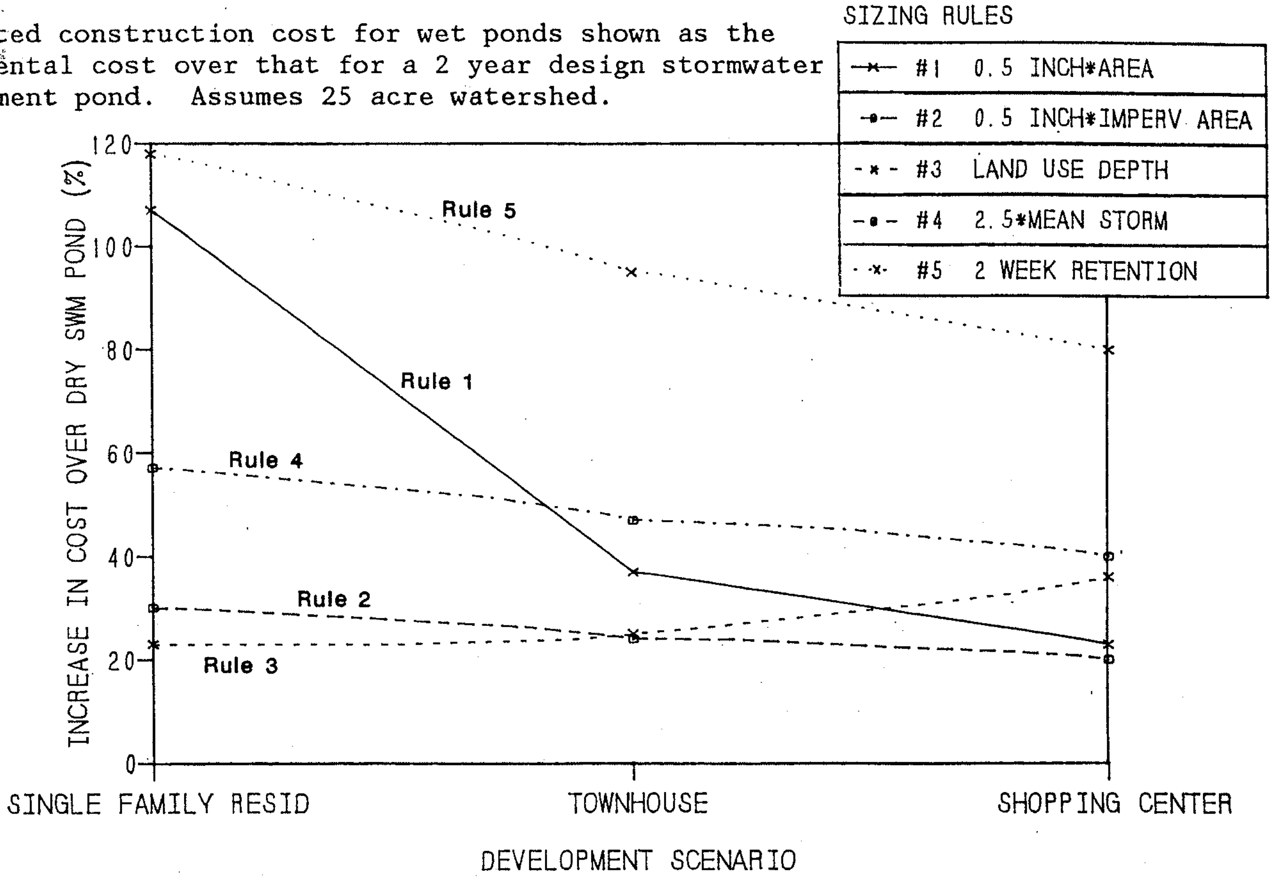
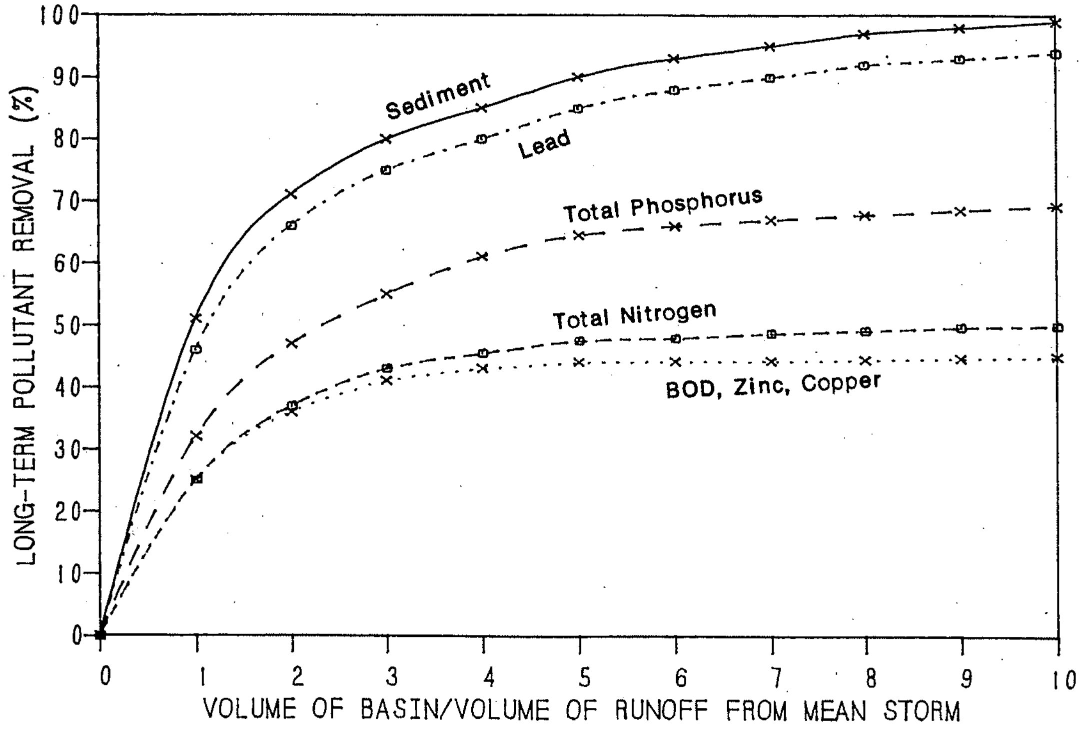


Figure 4.6: Estimated Removal of Selected Urban Pollutants as a Function of Permanent Pool Size



NOTE: Average results from U.S. EPA (1986), and adapted to reflect modifications of Walker (1986).

Table 4.1: Summary of Wet Pond Pool Sizing Rules

SIZING RULE	SEDIMENT REMOVED	PHOSPHORUS REMOVED	EXTRA STORAGE (compared to 2 year dry pond)	EXTRA COST (compared to 2 year dry pond)
RULE 1: 0.5 inch runoff per acre	60-90%	35-90%	35-200%	20-90%
RULE 2: 0.5 inch runoff per impervious acre	60%	35-40%	30%	20-25%
RULE 3: 0.1 to 0.8 inches depending on land use	55-80%	30-50%	30-70%	20-40%
RULE 4: 2.5 times the runoff of the mean storm	75%	55%	75%	40-50%
Rule 5: 4.0 times the runoff of the mean storm (≈2 week retention)	85-90%	65%	200-250%	80-100%

Pond Shape

Short-circuiting is a frequently cited problem in wet pond design, whereby incoming runoff passes through the pond without displacing the old water. (Schaefer, 1986). Short-circuiting can be largely prevented by maximizing the distance between the pond inlet(s) and outlet. For this reason, many local stormwater management ordinances specify minimum length to width ratios of 3:1 or greater. If local topography makes the excavation of a long, narrow pond impossible, baffles or gabions can be placed within the pond to lengthen the flow-path between the inlet and outlet.

Long, narrow and irregular shapes are also desirable for shallower ponds since they reduce surface area exposed to the wind and thereby prevent resuspension of previously deposited materials (Schaefer, 1986). Irregularly shaped ponds also have a less "engineered" appearance, and produce a more natural landscaping effect.

Pond Depth

Since much of the pollutant removal in a wet pond is accomplished by gradual settling, pond depth is an important design aspect. Settling column studies and modeling analyses indicate that shallow ponds have higher removal efficiencies than deeper ones. However, extremely shallow ponds (< 2 feet in depth) may be prone to resuspension problems, caused by wind generated disturbances of bottom sediments. Therefore, shallow ponds should be avoided unless they are stabilized by aquatic vegetation. On the other hand, pond depths in excess of 8 feet should also be avoided to prevent the onset of thermal stratification. Stratified ponds tend to become anoxic more frequently than shallower ponds, have non-ideal settling characteristics, and may release previously deposited pollutants from the sediment back to the water column. For design purposes, an average pond depth of 3 to 6 feet should be optimal.

As shown in the schematic wet pond design (Figure 4.1), the depth of the permanent pool should be variable. For example, a minimum 10 foot wide, one foot deep shelf is needed around the perimeter of the pond to provide suitable conditions for the establishment of aquatic vegetation, and to reduce the potential safety hazard to the public. Shallow depths near the pond inlet may be required to concentrate sediment deposition in a smaller, more easily accessible area. Pockets of deeper water (6 feet +) are necessary as habitat refuges for fish, if the pond is managed for recreation. Normally, the riser should be located in a deeper area within the pond to accommodate coldwater bottom release if thermal impacts need to be mitigated.

Aquatic Vegetation

Establishment of aquatic vegetation around the perimeter of a wet pond enhances pollutant removal, and also has several other beneficial uses. Emergent plants such as bulrush, three-square and lizards tail can provide an attractive fringe habitat, providing food and cover for wildlife and waterfowl. The marsh fringe also protects the shoreline from erosion, and if situated near the inlet to the pond, can trap incoming sediment. While most emergent plants withdraw nutrients from the sediments rather than the water column, associated algae which are attached to the plants, or grow nearby on the shallow sediments, are capable of soluble nutrient removal. Shallow, organic-rich waters in the marsh fringe provide an ideal environment for bacteria and other microorganisms that reduce organic matter and nutrients. Similarly, the marsh fringe provides a habitat for predacious insects that can serve as a natural population control for mosquitos and other nuisance insects. From an aesthetic standpoint, the fringe of aquatic vegetation conceals trash and floatable debris and disguises and stabilizes the pond shoreline (which is often barren due to fluctuating water levels). Appropriate wetland plant species and propagation techniques are described in the basin landscaping guide in Chapter 9.

Side-slopes

The slopes leading to the wet pond should be gradual to prevent erosion of the banks. Dense vegetation is hard to establish and maintain on steeply sloping banks. In the absence of vegetative cover, gully erosion may actually make the pond a sediment source. Most local stormwater management guidelines suggest that side-slopes be no greater than 3:1 (h:v), and preferably flatter. Banks steeper than 3:1 should be stabilized with riprap to prevent erosion. Low slopes make routine mowing of the banks easier and safer, and allow better pond access for maintenance purposes. Adams et al. (1983) have demonstrated greater wildlife and waterfowl utilization of ponds with gentle side-slopes in comparison to steep side-slopes, and recommend 10:1 (h:v) slopes where feasible. The risk of a child stumbling into a wet pond is also presumably minimized with low sloping banks. Naturally, there is a trade-off in having extremely flat side-slopes, since steeply sloped banks provide more stormwater capacity per surface area.

Inlet and Outlet Protection

The stream channel immediately below the pond outlet should be lined with large stone riprap to prevent scouring and have a slope close to 0.5% (MNCPPC, 1984). A layer of filter cloth should be laid down first to conform to the natural dimensions of the channel, and then anchored with 18 to 30 inch stone riprap. If the outfall pipe diameter is less than 24 inches, smaller sized 9 to 12 inch riprap can be used. Structural measures such as stilling basins can also be employed to reduce the runoff velocity from the pipe outfall.

The invert elevation for all inlet pipes should be set to discharge at or below the surface of the permanent pool. Pipes which outfall above the pool are not acceptable as they erode the banks and side-slopes of the pond. Inlet pipes should be located within a foot of the permanent pool elevation.

PHYSICAL SUITABILITY AT THE SITE LEVEL

Minimum Drainage Area

Construction of wet ponds is not generally feasible in watersheds less than ten acres in size, unless a natural spring occurs on-site (infiltration basins can be used as an alternative). Maintenance of a permanent pool is difficult in these small watersheds because infiltration and evaporation losses will often result in severe pond drawdowns. Baseflow in small watersheds often runs out during the summer months and cannot compensate for the gradual drawdown. Consequently, pond stagnation may result in algal matting and odor problems. During extremely dry periods, the pond may completely dry up. While none of these factors reduces the pond's capability to remove pollutants (and may actually enhance them to some degree), it is not likely that residents will be pleased to live near such an unattractive nuisance. For these reasons, infiltration basins or extended detention ponds are a more suitable alternative on very small development sites.

The feasibility of constructing wet ponds on moderate sized watersheds (10 to 30 acres) should be checked using the procedures set forth in Md WRA (1986c). A general rule of thumb is that four acres of contributing watershed are needed for each acre-foot of storage (Md SCS, 1976).

Permeable Soils

Severe pond drawdowns may occur even in larger wet ponds if pond soils are permeable (hydrological soil groups "A" and "B"), or, if they extend into fractured bedrock. Drawdowns can be minimized by installing a six inch liner of clay soil, filter fabric, or merely by compacting the pond soils (Md WRA, 1986c). In most areas of the Piedmont, clay soils (minimum 25% clay by weight) can be scavenged from other areas of the development site during the grading process.

Depth to Bedrock

If the bedrock layer lies close to the surface of the soil, it may become too difficult or expensive to excavate needed storage for an extended detention pond. Soil maps should be consulted, and soil borings need to be taken to confirm that no bedrock needs to be excavated.

Land Requirements

Wet ponds are probably not a feasible BMP in watersheds where land costs or space are at a premium. In small watersheds, the pond and its buffer can consume as much as 10% of the watershed area, particularly if a generous pond sizing rule is used. In most cases, however, the pond and its buffer will consume much less than 5% of the total watershed area. Most local governments have adopted open space requirements for new developments which usually reserve more than sufficient land for siting a pond.

Utility Relocation

Most utility companies will not allow existing underground pipes to be submerged under a permanent pool of water (as this can lead to infiltration/ inflow problems and make maintenance efforts extremely difficult). Therefore, if wet ponds are to be used, the site designer should check to see if the pool area will cross any utility right of way.

Wetland Permits

Often, the best location to put a pond in a site is in low marshy areas and natural depressions. Unfortunately, these areas are often classified as freshwater wetland habitat, and may be protected under state or federal wetland law. It is important to note that many wetland habitats are not easy to identify. The designer should consult local wetland maps and wetland permitting agencies to determine if the area has wetland status. If so, permits must be secured. See Herson (1987) for a summary of local wetland requirements and the permitting process.

WET POND COSTS

Predicting Wet Pond Costs

A planning estimate of the base construction cost for a wet pond of less than 100,000 cubic feet of storage can be approximated using the MWCOG equation (Wiegand et al., 1986):

(EQ 4.1) $C = 6.1Vs^{0.75}$

where C = construction cost in 1985 dollars.
Vs = volume of storage (cubic feet) of the pond up to the crest of the emergency spillway, including the permanent pool.

Similarly, first-cut cost estimates for larger wet ponds (greater than 100,000 cubic feet storage) can be derived from the following:

(EQ 4.2) $C = 34Vs^{0.64}$

As an example, if a planned wet pond were to have a permanent pool of 12,000 cf and additional stormwater storage of 30,000 cf, the estimated total cost of the BMP would be 6.11(42,000)**.75 or about 18,000 dollars. The actual cost at a site will vary around this value, depending on the degree of excavation required. Costs can be significantly lowered if natural depressions and topography are creatively used.

Both equations only estimate the cost of constructing a wet pond. Land costs are not considered because of their great variability. In most cases, the assumption of zero land costs is reasonable since most zoning boards require that a minimum percentage of site area be reserved for open space. Additional contingency costs associated with designing the pond, securing the necessary permits, and overseeing construction can be estimated using a rule of thumb that these costs generally add 25% to the base construction cost (C).

As the form of the equations indicate, wet pond construction costs are largely determined by the total storage volume. Since the stormwater component of the total storage volume is fixed, the incremental cost of wet ponds over dry ponds is a function of permanent pool size. Thus, the sizing rule selected for the permanent pool will strongly influence the total cost. Figure 4.4 shows the impact of each pond sizing rule on wet pond construction cost. In most cases, the unit cost of wet ponds declines as development intensity increases. Likewise, the fact that the exponent in the equation is less than one indicates the presence of economies of scale, such that wet ponds are relatively cheaper when applied to larger watershed areas than to smaller ones.

Cost-Effectiveness

Previous studies have shown that the most cost-effective application of wet ponds is in larger and more intensive development sites (Schueler et al., 1985). Because of this, wet ponds are good candidates for regional stormwater management, whereby the runoff from several development sites is treated in one central facility. Wet ponds are not the most cost-effective BMP option in smaller residential sites, where extended detention or infiltration basins/trenches may be more economical. The most cost-effective wet pond designs are generally 30-60% more expensive than a dry pond of similar stormwater capacity (Figure 4.4).

Other Economic Factors

In addition to removing urban pollutants, wet ponds also provide several urban amenities of economic value. These include aesthetics, recreation, wildlife habitat, and landscape value. While it is often impossible to assign these an exact dollar value, there is ample documentation that homes located near well designed and maintained wet ponds command higher prices than homes not so located (Baxter et al., 1985; Adams et al., 1983) or homes situated near less attractive BMPs such as dry ponds. Higher land values may compensate for the greater incremental cost of constructing a wet pond. Numerous opinion surveys have indicated that a majority of homeowners are willing to pay more for a home situated near a naturally landscaped pond managed for wildlife (Adams et al., 1983). An implication of this finding is that modest extra investments in designing and landscaping wet ponds will probably be recouped in the form of higher housing prices.

MAINTENANCE REQUIREMENTS

A clear requirement for wet ponds is that a firm institutional commitment be made to carry out both routine and non-routine maintenance tasks. The nature and cost of wet pond maintenance requirements are outlined below, along with design tips that can help to reduce the maintenance burden.

Routine Maintenance

MOWING

The side-slopes, embankment and emergency spillway of a wet pond must be mowed at least twice a year to prevent woody growth and control weeds. More frequent mowing may be demanded in residential areas by adjacent homeowners concerned about neighborhood appearance or allergies. This usually entails about 14 mowing operations annually, and constitutes the largest routine maintenance expense. The use of native or introduced grasses which are water tolerant, hardy and slow-growing is recommended. Some representative species, such as K-31 Tall Fescue and Crown Vetch, are described in the basin landscaping guide in Chapter 9.

INSPECTIONS

Wet ponds need to be inspected on an annual basis to ensure that the structure operates in the manner originally intended. When possible, inspections should be conducted during wet weather to determine if the pond is functioning properly. Inspection priorities should include checking the embankment for subsidence, erosion, cracking, and tree growth; the condition of the emergency spillway and drain; the accumulation of sediment, clogging of the barrel and outlet; the adequacy of upstream and downstream channel erosion protection measures; any modifications which have occurred to the contributing watershed and the pond structure; and the stability of the side-slopes. Inspections should be carried out with as-built pond plans in hand.

DEBRIS AND LITTER REMOVAL

As a part of periodic mowing operations, debris and litter should be removed from the surface of the pond. Particular attention should be paid to floatable debris around the riser, and the outlet should be checked for possible clogging.

EROSION CONTROL

The pond side-slopes, emergency spillway and embankment all may periodically suffer from slumping and erosion. Corrective measures such as regrading and revegetation may be necessary. Similarly, the riprap protecting the channel near the outlet may need to be repaired or replaced.

NUISANCE CONTROL

Most public agencies surveyed indicate that control of insects, weeds, odors, and algae may be needed in some problem ponds. Indeed, nuisance control is probably the most frequent maintenance item demanded by local residents. If properly sized and vegetated, these problems should be rare in wet ponds except under extremely dry weather conditions. Biological control of algae and mosquitos, by the use of fathead minnows and other fish, is preferable to chemical applications.

Non-Routine Maintenance

STRUCTURAL REPAIRS AND REPLACEMENT

Eventually, the various inlet/outlet and riser works in a wet pond will deteriorate and must be replaced. Some local public works experts have estimated that corrugated metal pipe (CMP) has a useful life of about 25 years, while concrete barrels and risers may last from 50 to 75 years (MNCPPC, 1985) No stormwater management pond has been in the ground for longer than twenty years in the Washington, D.C. area, so there is little local experience in this area. However, since the various water works constitute about 25% of the initial construction cost (Wiegand et al., 1986), their replacement can be a significant future expense.

Some ponds that suffer from excessive and chronic drawdowns often may have problems with leakage or seepage of water through the embankment. Corrective measures can be difficult, but can be avoided if the embankment has been compacted, and if anti-seep collars are used around the barrel.

SEDIMENT REMOVAL

If properly designed, wet ponds will eventually accumulate enough sediment to significantly reduce storage capacity of the permanent pool. As might be expected, the accumulated sediment can reduce both the appearance and pollutant removal performance of the pond. The best available estimate is that approximately one percent of the storage volume capacity associated with the two year design storm can be lost annually. Smaller, stabilized watersheds accumulate sediment at lower rates, while larger watersheds with unprotected channels or ongoing construction fill in more rapidly. (A planning estimate of the likely sediment accumulation at a site can be rapidly made using the Simple Method outlined in Chapter 1, Example 1-2).

A sediment clean-out cycle of ten to twenty years is frequently recommended in the Washington, D.C. metropolitan area (APWA, 1981; MWCOG, 1983b). The costs associated with each cycle of sediment removal can be staggering. One-time operations in excess of $100,000 are not uncommon in larger wet ponds and urban lakes. A review of several recent pond dredging projects in suburban Northern Virginia indicated that the average dredging cost was over $14 per cubic yard (cy), with a range of $6.25-22.40/cy (Wiegand et al, 1986). The variation in these costs is due to differences in the size and accessibility of the pond, the proximity of the disposal site, and the method used to remove and transport sediment. Costs for smaller wet ponds (Vs< 100,000 cf) typically range from $5-10/cy since sediment can be mechanically removed with a front-end loader after the basin is de-watered. Larger ponds normally require the use of the more expensive dragline or hydraulic dredge methods. Also, higher costs are incurred in transporting dredged materials to a suitable on-site disposal area, and in the grading and reclamation of the site after dredging has been completed.

Sediment removal costs become even higher when on-site disposal areas are not available. Hauling increases costs by $5-10/cy, depending on distance traversed. If dredged sediments are land-filled, tipping fees will increase removal costs by another $15-25/cy, depending on the jurisdiction.

Total Maintenance Costs

The annual cost for routine maintenance averages about $300-500 per maintained acre (which includes the pond and the surrounding buffer, and as a rule of thumb is oftened estimated at three times the surface area of the pond). Annual costs for non-routine maintenance (mainly sediment removal) are estimated to range from 1-2% of the pond's base construction cost. Therefore, it is recommended that homeowners and public works agencies budget 3-5% of the base construction cost of the wet pond annually to cover the routine and unexpected maintenance expenses.

Design Tips to Reduce Maintenance Costs

1. For easier mowing, side-slopes should be no steeper than 3:1 and no flatter than 20:1. The first guideline allows for easier access and safety, while the latter guideline acts to prevent soggy conditions.

2. Mowing costs can be reduced if the pond buffer is managed as a meadow rather than a lawn, as the frequency of mowing operations can be reduced from 14 to 2 times per year.

3. Hoods or trash racks should be installed on both the low flow and design storm orifices to prevent clogging. The low flow orifice pipe should be negatively sloped so that it draws water at least one foot below the surface of the permanent pool (see Chapter 3, Figure 3.3b).

4. Leakage through the embankment can often be prevented by using anti-seep collars around the barrel, and by compacting the embankment.

5. Reinforced concrete pipes, barrels and risers should be utilized because of their greater longevity (MNCPPC, 1985). The use of corrugated metal pipe should be kept to a minimum.

6. Where possible, the riser should be located within or on the face of the embankment rather than out in the middle of the pool. This makes the riser easier to maintain and inspect, visually pleasing, and also prevents floatation problems.

7. Extra fill should be placed on the pond embankment to account for future settling or subsidence. An allowance of 10-15% is often used.

8. All ponds should have an emergency drain (with the pipe sized to completely drain the pond in less than 24 hours), to allow access for riser repairs and heavy equipment needed for sediment removal.

9. Maintenance access must be provided to the pond from public or private right of way with a minimum width of 10 feet and maximum slope of 5:1 (h:v). Lack of proper access to ponds can lead to difficult and costly disputes over residential property damage in the future. The access road should never cross the emergency spillway.

10. On-site disposal areas capable of receiving sediment from at least two clean-out cycles should be reserved in adjacent open space, if possible. The size of the required disposal area can be roughly calculated as follows:

EXAMPLE 4-1: CALCULATING THE AREA NEEDED FOR ON-SITE SEDIMENT DISPOSAL

Step 1. Use the Simple Method in Chapter 1 to determine long-term sediment load from the upland watershed.

Step 2. Estimate wet pond trapping efficiency from Figure 4.2.

Step 3. Compute the volume of sediment trapped in the pond, assuming one ton equals a cubic yard of wet sediment.

Step 4. Solve for area assuming the disposal area can accept a 12 inch depth of wet sediment per unit area.

11. Extra storage, in the form of a sediment forebay, should be provided near the inlet to trap incoming sediments. This represents an extremely cost-effective means of reducing sediment removal costs, because dredging a cubic yard of sediment after a pond is built is at least five times more expensive than the cost of excavating it during construction. Shallow sediment forebays with aquatic plants are an attractive option, since they enhance the sediment trapping, pollutant removal, and concentrate accumulated sediment in an area where it can be readily removed without having to drain the entire permanent pool. The minimum volume required for a sediment forebay can be provisionally calculated using Steps 1 to 3 of the disposal area procedure outlined above.

12. The responsibilities for both routine and non-routine wet pond maintenance need to be clearly vested so that funds can be budgeted for a

regular maintenance program. If the responsibilities fall to a homeowners association, the nature and extent of their obligations should be clearly delineated in a legally binding agreement or covenant. Even if a public agency is not responsible for maintenance, private maintenance efforts should be monitored as a normal part of the inspection process. Because of the limited financial reserves and technical expertise of homeowners associations, public maintenance is clearly preferred.

ENVIRONMENTAL ATTRIBUTES OF WET PONDS

A wet pond represents a significant modification to the urban landscape, and has both positive and negative impacts on the natural and human environment. These impacts need to be carefully assessed during the site review stage to ensure that a pond is an appropriate choice.

Impacts on the Natural Environment

When properly designed and managed, wet ponds are an attractive habitat for fish and wildlife. Adams et al. (1983) have documented high wildlife and waterfowl utilization in wet ponds in Columbia, Maryland, particularly in comparison to adjacent dry ponds. Several local stocking programs have also shown that several species of warmwater fish can thrive in larger and deeper wet ponds. The value of wet pond habitat is high because of the general lack of quality habitat in urban areas and its close proximity to local residents.

On the other hand, wet ponds can severely disturb the sensitive ecology of headwater streams. The complex downstream impacts to aquatic life caused by stream regulation have been extensively studied (Ward and Stanford, 1979). For example, wet ponds heat up rapidly during the warmer months and increase downstream water temperatures by as much as 10-11°F (Galli, 1986). In a recent Maryland study, maximum water temperatures of 85°F were recorded in the discharge from a wet pond. This temperature exceeds most coldwater fishery standards in Maryland and Virginia, and perhaps more importantly, can severely stress and even kill sensitive fish populations. The problem is most pronounced in wet ponds situated in narrow, deep valleys that are not well mixed by the wind.

Wet ponds have been prohibited in the upper Paint Branch watershed in Montgomery County, Maryland which supports a native brown trout population because of concerns over thermal discharges (CH2M-Hill, 1980). Unfortunately, few other headwater streams in the region support coldwater fisheries due to past watershed and riparian disturbances. Even so, thermal discharges from wet ponds probably will stress the more tolerant and degraded stream biota that exists in most parts of the region today. Thermal discharges can be mitigated, to some extent, by bottom water releases and deep, over-sized pond designs. Planners should be sure to consult with state fish biologists when reviewing proposed wet pond applications.

Dissolved oxygen can become depleted periodically during the summer in wet ponds. This may result in the release of anoxic waters downstream (Galli, 1986; Free and Mulamootil, 1983). Natural reaeration processes will normally allow dissolved oxygen levels to recover within a few hundred yards below the pond. However, aquatic life within this short "mixing zone" may be drastically altered.

While wet ponds manage post-development peak discharges, they typically do not control increases in the frequency of bankfull discharges or the decrease in summer low flows. Both factors can degrade the quality of downstream aquatic habitat (see Chapter 1).

Concern has also been expressed about the impacts of wet ponds on upstream aquatic life, particularly when large regional structures are constructed. In these instances, the channel network conveying runoff to the pond is not protected from bank erosion, which may reduce the value of aquatic habitats.

Two points should be kept in mind when evaluating the overall impact of wet ponds on the local environment. First, while wet ponds may have potentially severe downstream impacts on aquatic life, it is also likely that impacts of urbanization may produce the same result. Preservation of sensitive aquatic environments in urbanizing areas requires a comprehensive watershed protection program including land use control, sediment control enforcement, land acquisition, monitoring and critical review of each new development proposal (CH2M-Hill, 1980). Unless these commitments are made, it is probable that downstream habitat will become degraded with or without wet ponds.

Design Tips to Improve Wet Pond Wildlife Habitat

The following management guidelines have been adapted from various wildlife management documents:

1. Aquatic vegetation which provides food and cover for wildlife should be established in the pond. General procedures for selecting and propagating wetland plants which have wildlife or waterfowl value are described in Chapter 9. Additional technical assistance can be obtained from cooperative extension agents.

2. A buffer strip with a minimum width of 25 feet around the pond should be planted with shrubs, trees and grasses, preferably those that provide food and cover for wildlife. The strip should not be continuous, since some species need bare loafing areas. No trees should be planted on the embankment. Adams et al. (1983) recommend that no more than 50% of the perimeter of the pond be heavily vegetated if waterfowl are the desired management objective.

3. The depth of the pond should be variable with at least 25% of the pond less than two feet deep. This objective can usually be met by a 10 to 20 foot aquatic bench around the perimeter and a shallow sediment forebay near the pond inlet.

4. Deeper areas (>5 feet) and large surface areas (>1 acre) may be required to maintain fish populations (SCS, 1986).

5. Baseflow must be present to maintain water levels near the design elevation in the summer months. Wet ponds located in watersheds of less than 20 acres should have a reliable water source and a clay liner.

6. Side-slopes should be gentle to promote wildlife and waterfowl utilization. If feasible, slopes should be graded to 10:1 (h:v).

Design Tips For Mitigating Downstream Impacts

1. Thermal discharges from wet ponds may be alleviated by designing risers to withdraw water from the bottom of the pond where water will normally be cooler. If bottom water releases are contemplated, it is often advisable to place surge stone in the barrel to help re-aerate the low oxygen bottom waters before they are discharged. Trees can also be planted (or saved during site clearing) around the perimeter of the pond to provide shade that prevents rapid warming. Additionally, ponds can be located in exposed areas where prevailing winds can help to mix the waters of the pond, thereby reducing the stratification in the pond.

2. Aerators or fountains can be installed in ponds to maintain dissolved oxygen levels. Relatively cheap wind-driven aerator models are now available. A well-anchored riprap "cascade" can be incorporated into the outfall of the pond for better reareation.

3. Detention times in wet ponds can be extended to provide greater downstream bank erosion control at little additional expense (see Chapter 3).

4. Various stream improvement techniques can be applied to mitigate downstream habitat degradation, such as log check dams, rock or log deflectors, and vegetative or riprap protection of channels. An excellent state of the art review of stream improvement technologies is provided in Wesche (1985).

5. Additional mitigation measures may be suggested by fisheries and wildlife professionals that are consulted during the design phase.

Impacts on the Human Environment

Resident surveys indicate that wet ponds are a popular BMP with the public, if they are well designed and maintained. Both Adams et al. (1983) and Tassone (1984) found that residents preferred wet ponds over dry ponds by a three to one margin in separate surveys in Maryland. Residents feel that ponds enhance property values, add to the appearance of the community, and promote a sense of community (Baxter et al., 1985). Deeper, vegetated ponds that are managed for wildlife were clearly preferred by residents. Some respondents complained about safety, mosquitos, odor, turbidity, and algae problems, but on the whole, most felt that the beneficial aspects of wet ponds outweighed the temporary nuisances.

Most of the residents surveyed recognized the need for frequent maintenance of ponds in the surveys. Several larger homeowners associations in the region (Columbia, Maryland; Reston, Virginia; and Montgomery Village, Maryland) have gone so far as to adopt watershed and pond management/maintenance programs in recent years.

Concerns have been raised about the safety aspects of wet ponds, particularly with respect to liability. A few jurisdictions do require that wet ponds be fenced to prevent access by children; most, however, feel that fences are an "attractive nuisance". Wet ponds can be designed to minimize the risk of accidental drowning by keeping them relatively shallow, installing an underwater safety bench, avoiding any sharp drop-offs from shores, keeping sideslopes gentle, and fencing off large diameter outfalls.

RELEVANT DESIGN GUIDANCE

The design summary presented on the following pages provides a summary of some of the more important design features to consider when planning a wet pond. These are also shown in schematic form in Figure 4.7. In addition, the following references should be consulted when designing a wet pond:

Md. Soil Conservation Service. 1981. Standards and Specifications for Ponds. Practice Code No. 378.

Md. Association of Soil Conservation Districts. 1976. Stormwater Management Pond Design and Construction Manual.

Md. Water Resources Administration. 1986. Feasibility and Design of Wet Ponds to Achieve Water Quality Control.

U.S. Environmental Protection Agency. 1986. Methodology for Analysis of Detention Basins for Control of Urban Runoff Quality. EPA-600/2-80-135.

U.S. Soil Conservation Service. 1987. Revised Urban Hydrology for Small Watersheds. Technical Release No. 55.

DESIGN SUMMARY: WET PONDS

- SIZE:
  At a minimum, the volume of the permanent pool should be at least 2.5 times greater than the runoff volume generated by the mean storm (0.45 inches*runoff coefficient*watershed acres). The contributing watershed area should be at least ten acres in size, the pond surface area at least 1/4 acre.

- SHAPE:
  The pond should be wedge-shaped, narrowest at the inlet and widest at the embankment. A minimum length to width ratio of 3:1 should be used unless gabion baffles are used to extend the flow path. Irregular shorelines are preferred.

- DEPTH:
  Pond depth should average 3-6 feet, with a shallow underwater bench (minimum 10 feet wide) around the pond's perimeter. Water depths should be shallow near the inlet and deeper at the riser. Extremely deep ponds (average depth 8 feet or more) should normally be avoided to prevent stratification and to minimize excavation costs.

- SIDE-SLOPES:
  Side-slopes should be no steeper than 3:1 (h:v) and not flatter than 20 to 1. A flat safety bench, at least 10 feet wide, should be located near the toe of the slope.

- SOILS:
  If soils at the pond site are highly permeable, (in the "A" or "B" hydrologic groupings), it may be necessary to line the bottom of the pond with an impermeable geotextile or a six inch clay liner. Often, impermeable fill soils can be found elsewhere in the development for this purpose.

- RISER DESIGN:
  Where feasible, wet ponds should be designed to accommodate 24 to 40 hours of extended detention for a one-inch storm. Orifices used to maintain the permanent pool level should withdraw water at least one foot below the surface, and where environmental conditions warrant, close to the pond bottom. Hoods or trash racks should be installed on the riser to prevent clogging. For access and aesthetic reasons, the riser should be placed near or within the embankment. However, if the riser is located on the face of the embankment, fencing may be required. To reduce future maintenance requirements, concrete, rather than corrugated metal pipe, should be used for barrels and risers.

- OUTFALL PROTECTION:
  The channel immediately below the pond should be modified to conform to natural dimensions, and lined with large rip-rap placed over filter cloth. Stilling basins, rock deflectors, check dams and other devices should be used to reduce flow velocities to non-erosive levels.

- **POND BUFFER:**
  Wet ponds should be surrounded by a buffer strip, at 25 feet wide. The buffer strip should be planted with water tolerant, low maintenance grasses, shrubs and trees.

- **VEGETATION:**
  When feasible, it is suggested that artificial marsh fringe be established near the inlet or forebay, and around at least 50% of the pond perimeter.

- **EMBANKMENT:**
  At least 10-15% extra fill should be allowed on the embankment to account for possible subsidence. The embankment should have at least one foot of freeboard above emergency spillway. Anti-seep collars should be used to prevent seepage around the barrel, and a core trench installed under the embankment to key it to the substrate. The embankment should be graded to allow access, and mowed twice annually to prevent woody growth.

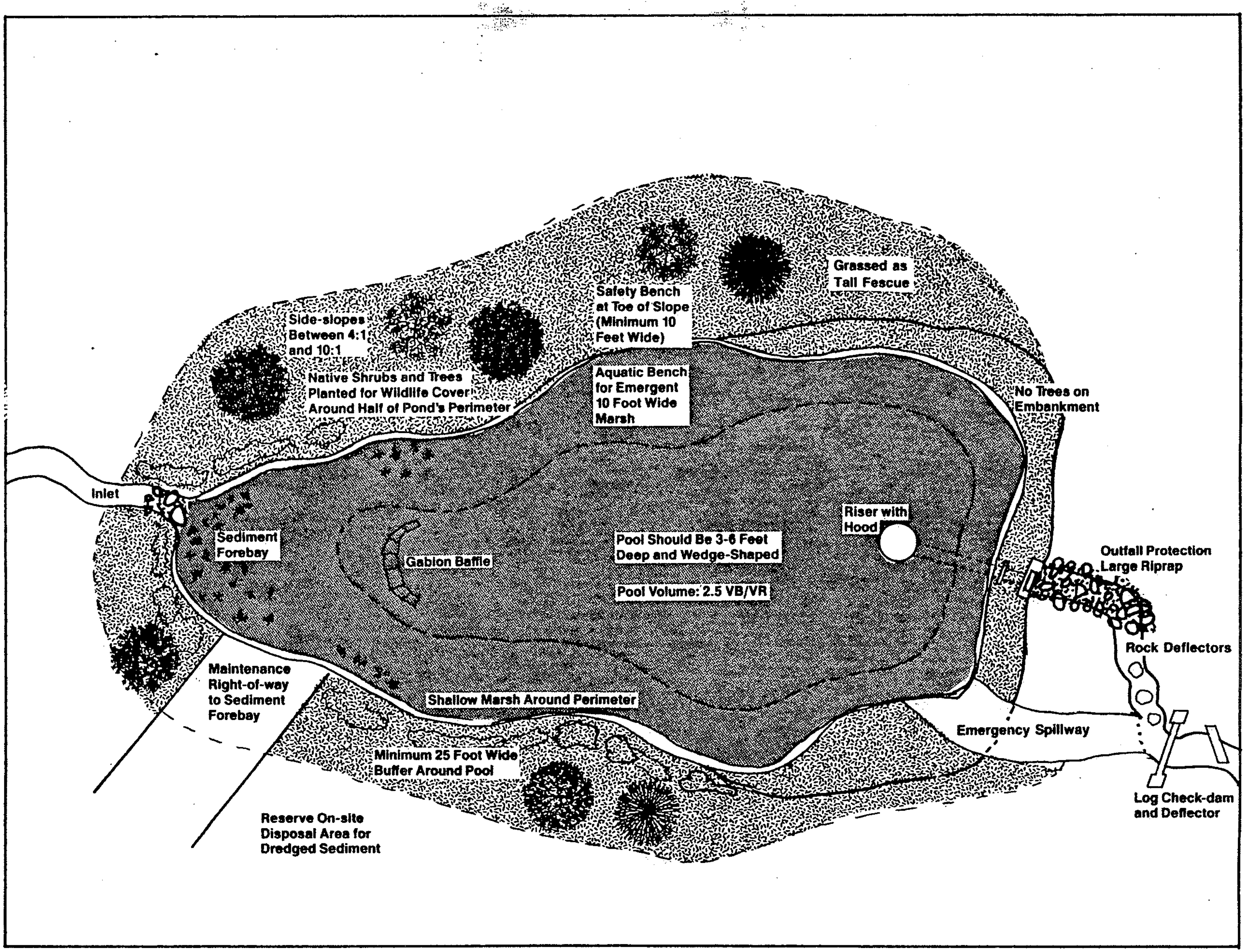
- **SITE ACCESS:**
  Adequate access from public or private right of way to the pond should be reserved. The access should be at least 10 feet wide, on a slope of 5:1 or less, and stabilized to withstand the passage of heavy equipment. The access road should not cross the emergency spillway, unless it is properly stabilized.

- **ENVIRONMENTAL REVIEW:**
  Consult with planners and biologists to develop a design that mitigates impacts to downstream aquatic life. Prepare a landscaping plan that provides habitat requirements for wildlife and waterfowl and is attractive to local residents.

- **MAINTENANCE:**
  Wet weather inspections should be conducted annually, with as-built plans in hand. Maintenance responsibilities should be clearly vested, with funds reserved for both routine and non-routine tasks.

- **SEDIMENT REMOVAL:**
  Begin final construction after upland area has been stabilized. Construct a sediment forebay near the inlet of the pond with extra storage equal to the volume of projected sediment trapping over a 20 to 40 year period. Reserve on-site sediment disposal areas near the pond in the form of a site easement.

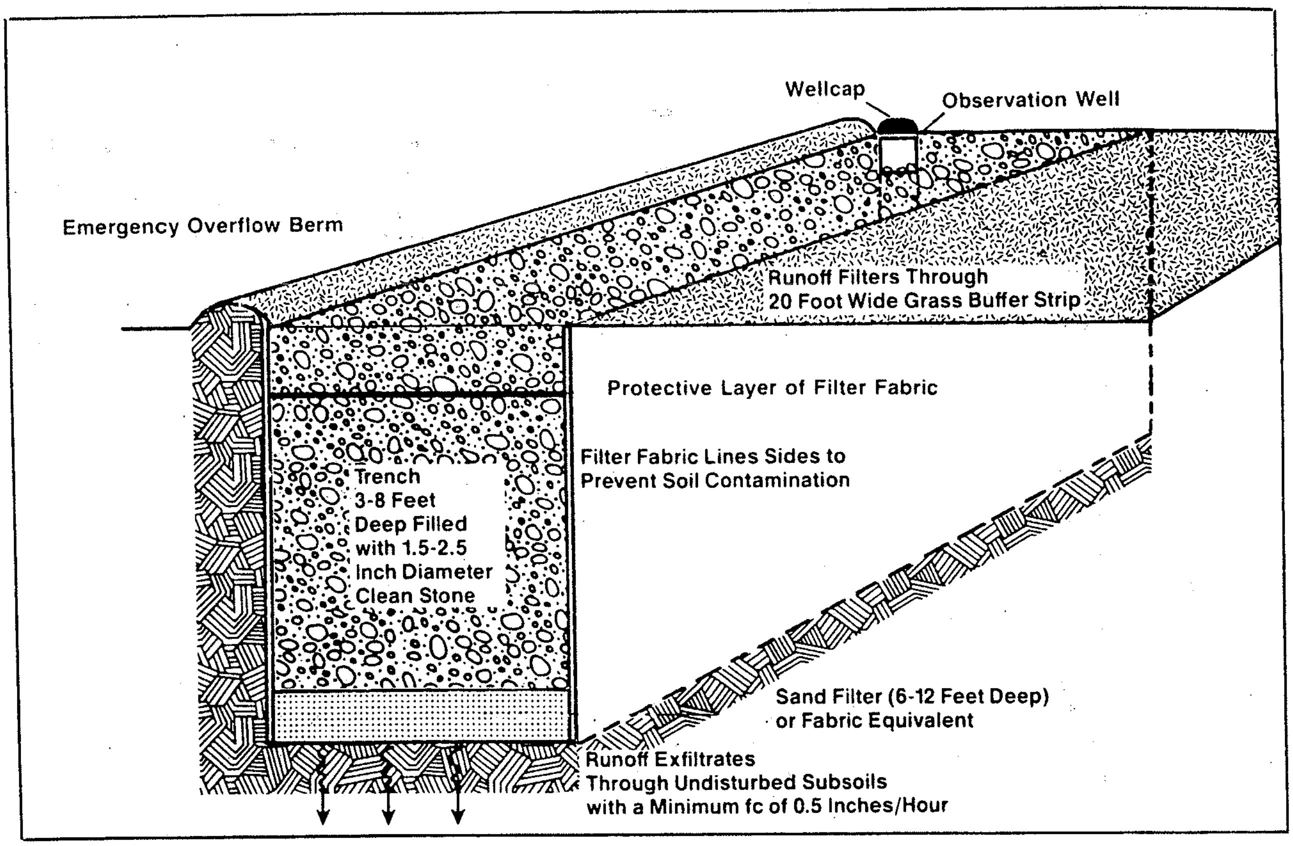
Figure 4.7: Design Schematic of a Wet Pond



CHAPTER 5: INFILTRATION TRENCHES

Infiltration trenches are an adaptable BMP that effectively remove both soluble and particulate pollutants. As with other infiltration systems, trenches are not intended to trap coarse sediments. Grass buffers (for surface trenches) or special inlets (for underground trenches) must be installed to capture sediment before it enters the trench. Depending on the degree of storage/exfiltration achieved, trenches can provide groundwater recharge, low flow augmentation and localized streambank erosion control. Individual trenches are primarily an on-site control, and are seldom practical or economical on sites larger than 5 or 10 acres. Trenches are only feasible when soils are permeable and the water table and bedrock are situated well below the bottom of the trench. Aside from regular inspections and more rigorous sediment and erosion control, trenches have limited routine maintenance requirements. However, trenches will prematurely clog if sediment is not kept out before, during and after construction of a site. If a trench does become severely clogged, partial or complete replacement of the structure may be required.

Figure 5.1: Schematic of an Infiltration Trench



Advantages of infiltration trenches are that they preserve the natural groundwater recharge capabilities of the site, are relatively easy to fit into the margins, perimeters and other unutilized areas of a development site, and are one of the few BMPs that provide pollutant removal on small sites or infill developments.

The disadvantages associated with infiltration trenches include practical difficulties in keeping sediment out of the structure during site construction (particularly if development occurs in phases), the need for careful construction of the trench and regular maintenance thereafter, and a possible risk of groundwater contamination.

INFILTRATION TRENCH METHODS

A schematic of an infiltration trench is shown in Figure 5.1. Basically, runoff is diverted into a shallow (3-8 feet deep) excavated trench that has been backfilled with stone to form an underground reservoir. Runoff is then either exfiltrated from the reservoir into the underlying subsoil or is collected by perforated underdrain pipes and routed to an outflow facility.

Trench size depends on two factors: the volume of runoff controlled, and the degree to which exfiltration is used to dispose of runoff. Typically, larger trenches are needed for stormwater control, whereas smaller versions can be employed for water quality purposes. The three basic trench systems are described below.

Complete Exfiltration System

In this design, runoff can only exit the trench by exfiltrating through the stone reservoir and into the underlying soils (i.e., there is no positive pipe outlet from the trench). As a result, the stone reservoir must be large enough to accommodate the entire expected design runoff volume, less any runoff volume lost via exfiltration during the storm. The complete exfiltration system provides total peak discharge, volume, and water quality control for all rainfall events less than or equal to the design storm. A rudimentary overflow channel, such as a shallow berm or dike, may be needed to handle any excess runoff from storms greater than the design storm.

Partial Exfiltration System

It may not always be feasible or prudent to rely totally on exfiltration to dispose of runoff. For example, there may be concerns about the long-term permeability of the underlying soils, downstream seepage, or clogging at the interface between the filter fabric and subsoil.

Many current designs use a perforated underdrain at the bottom of the trench to collect runoff and direct it to a central outlet. Since trenches are narrow, the collection efficiency of the underdrain is very high. As a result, these designs may only act as a short-term underground detention system. The low exfiltration rates and short residence times, together, result in poor pollutant removal and hydrologic control.

Performance of partial exfiltration systems can be improved during smaller storms when perforated underdrains are not used. Instead, a perforated pipe can be inserted near the top of the trench (Figure 5.2). Runoff then will not exit the trench until it rises to the level of the outlet pipe. Storms with less volume than the design storm may never fill the trench to this level, and will be subject to complete exfiltration.

In either design, the passage of the inflow hydrograph through the trench can be modeled using the modified TR-20 procedure (Md WRA, 1983b) to determine the appropriate sizing of the trench. Due to storage/timing effects, partial exfiltration trenches will be smaller in size than full exfiltration trenches serving the same site.

Water Quality Exfiltration Systems

The storage volume of a water quality trench is set to receive only the first flush of runoff volume during a storm. The first flush volume has been variously defined as; 1) one-half inch of runoff per impervious acre, 2) one-half inch runoff per acre, and 3) the volume of runoff produced by a one inch storm. The remaining runoff volume is not treated by the trench, and is conveyed to a conventional detention or retention facility downstream.

While water quality exfiltration systems do not satisfy stormwater storage requirements, they may result in smaller, less costly facilities downstream. The smaller size and area requirements of water quality exfiltration systems allows considerable flexibility in their placement within a development site, an important factor for "tight" sites. Additionally, if for some reason, the water quality trench fails, stormwater may still adequately be controlled by a downstream SWM facility.

INFILTRATION TRENCH DESIGN VARIATIONS

Trench designs can be further distinguished as to whether they are located on the surface or below ground. Surface trenches accept diffuse runoff (sheet flow) directly from adjacent areas, after it has been filtered through a grass buffer. Underground trenches can accept more concentrated runoff (from pipes and storm drains), but require the installation of special inlets to prevent coarse sediment and oil/grease from clogging the stone reservoir. Several examples of surface and underground trench designs are shown in Figures 5.2 to 5.9, and are described below. In most cases, these designs are adaptable for either full, partial or water quality exfiltration.

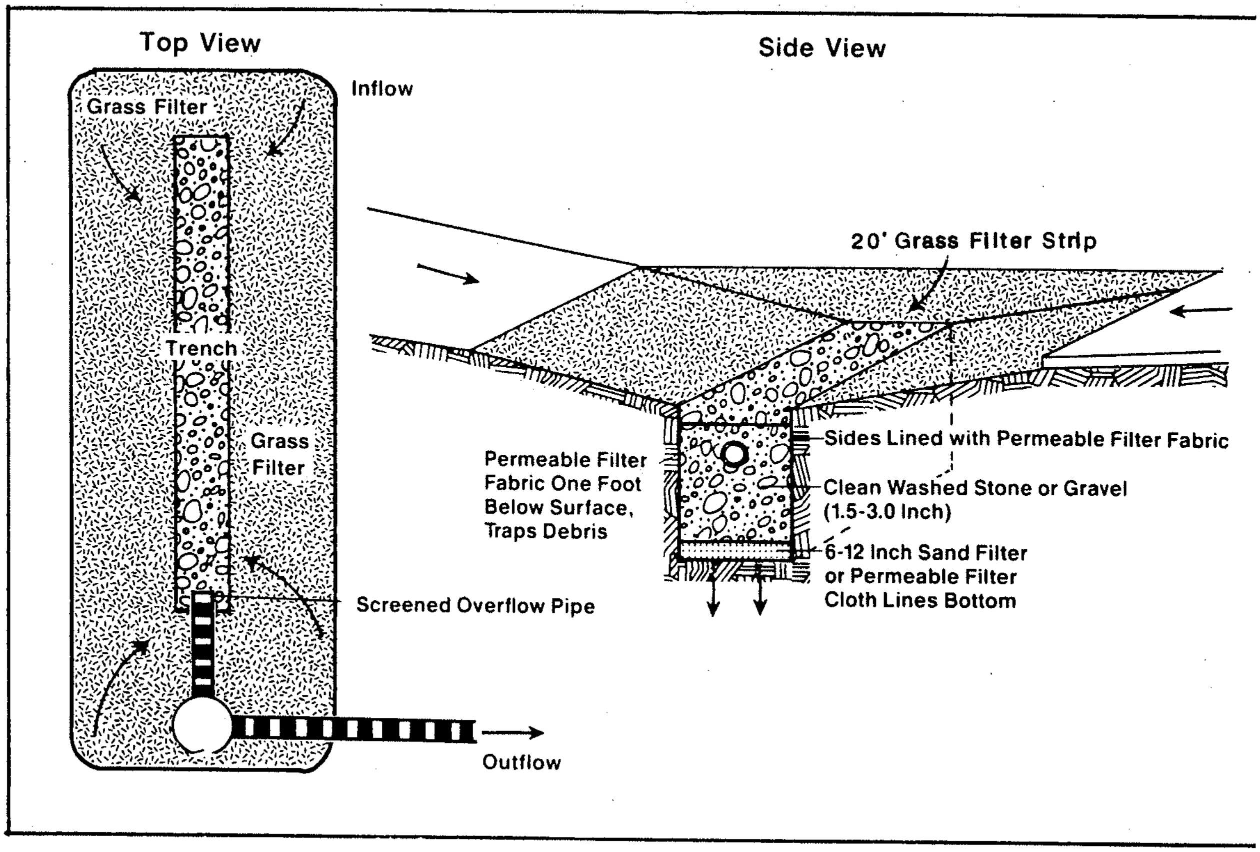
Surface Trench Applications

Surface trenches are typically applied in residential areas, where smaller loads of sediment and oil can effectively be trapped by grass filter strips. Since the surface is exposed, these trenches have a slightly higher risk of clogging than underground trenches. However, if preventative measures are taken (e.g., placing permeable filter fabric 6-12 inches below the surface of the trench to intercept sediment), any surface clogging that occurs can be relieved without having to reconstruct the entire trench. Because of their accessibility, surface trenches are easier to maintain and inspect.

DESIGN 1:

Median Strip Design (Figure 5.2). This design is frequently used for highway median strips and parking lot "islands" (depressions in between two lots or adjacent sides of one lot). Sheet flow is accepted from both sides of the trench, and is filtered through a 20 foot wide grassed buffer strip. The strip is an integral part of the trench, and should be graded to have a uniform slope not greater than 5%, and should directly abut the contributing impervious area. Berms located on each side of the strip form a shallow depression that temporarily stores runoff before it enters the trench. An overflow pipe is used to pass excess runoff.

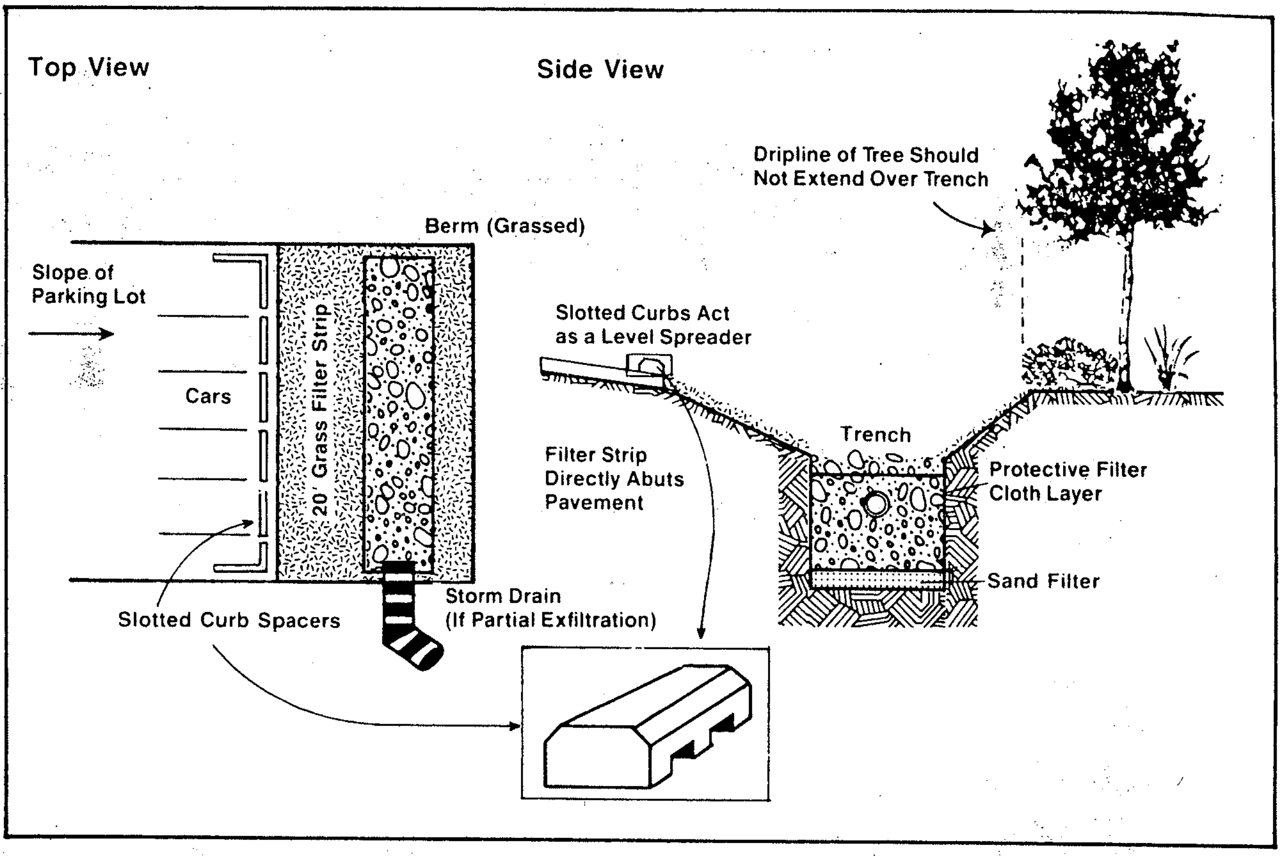
Figure 5.2: Median Strip Trench Design



DESIGN 2:

Parking Lot Perimeter (Figure 5.3). This design accepts sheet flow from the lower end of a parking lot. Slotted curb spacers are used as level spreaders to route sheet flow from the parking lot over the 20 foot wide filter strip (and also keep cars from damaging the strip). After being filtered over the grass strip, runoff enters the surface of the trench. A shallow berm is installed at the far end of the trench to ensure that runoff does not escape. The trench should have an overflow to pass large design storms, such as a PVC pipe with holes drilled on its underside, set near the top of the trench (Figure 5.3).

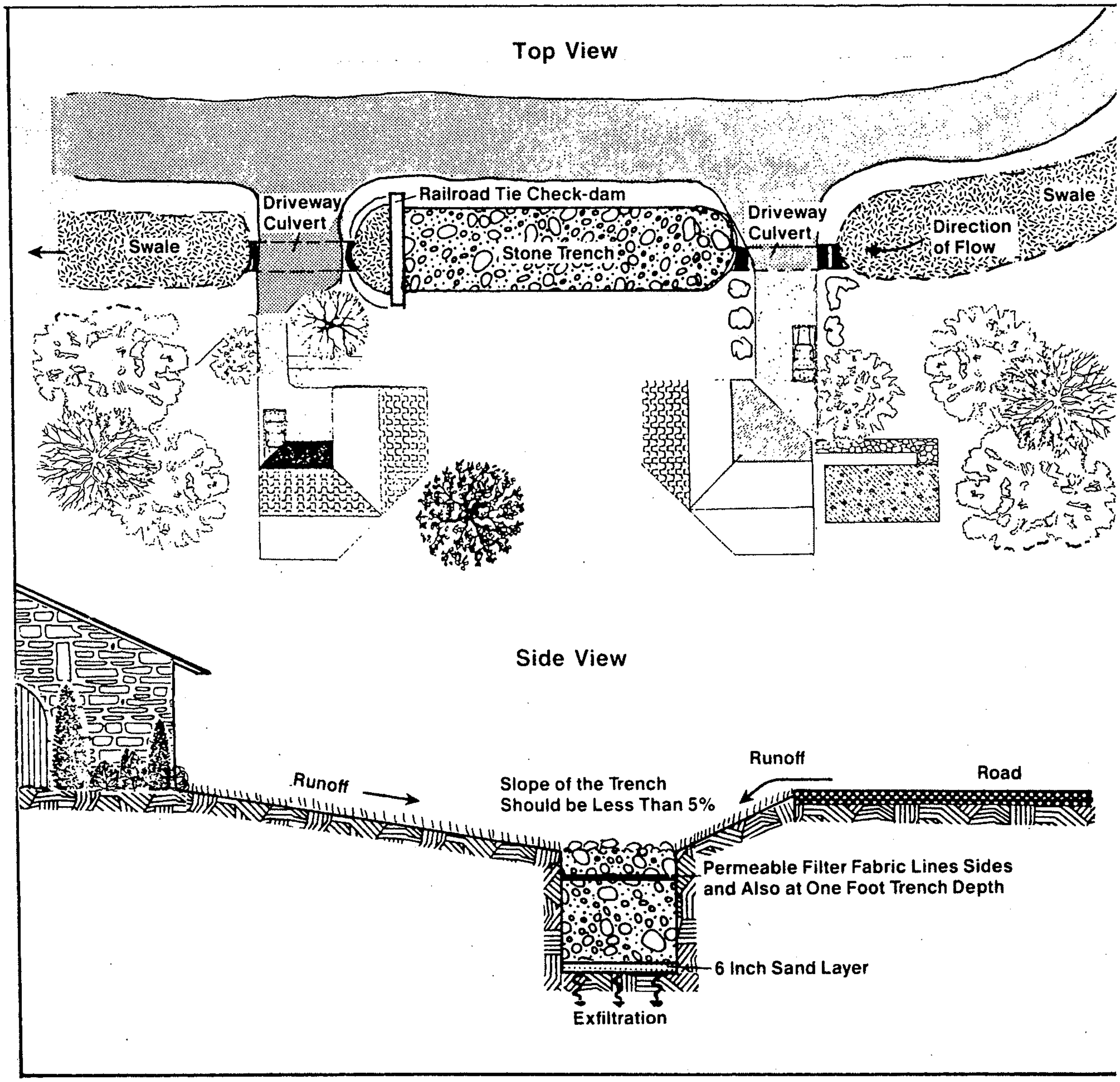
Figure 5.3: Parking Lot Perimeter Trench Design



DESIGN 3:

Swale Designs (Figure 5.4). Low density residential runoff (5-15% impervious) can be treated through a series of surface trenches located in swale drainage systems. The major design requirement is that the longitudinal slope of the swale collection system should never exceed 5%. Otherwise, concentrated flows will develop that might erode the swales and contaminate the trench. In addition, concentrated flows may pass around or over the surface of the trench and never infiltrate. An earthen check dam or railroad tie placed perpendicularly to the flow path, on the downstream side of the trench, can prevent "short-circuiting" and increase the volume of runoff exfiltrated by the trench. The slope of the trench should be as close to zero as feasible, and should have sideslopes of 5:1 (h:v) or less.

Figure 5.4: Swale/Trench Design



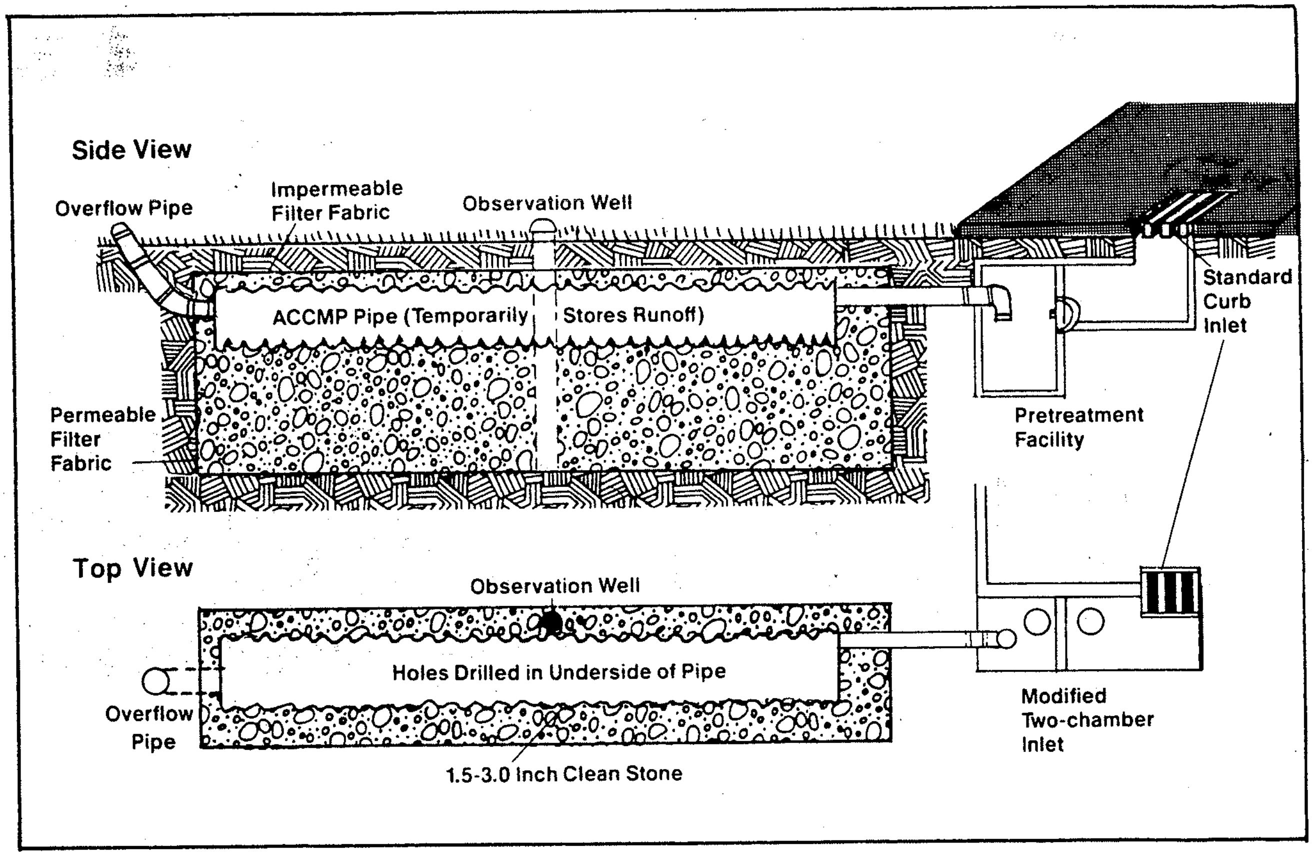
Underground Trench Applications

Underground trenches can be applied in a variety of development situations, and are particularly suited to accept concentrated runoff. However, it is important to pretreat concentrated runoff before it enters underground trenches, and to evenly distribute it within the trench. The top of the trench is protected by a layer of impermeable geo-textile, and is covered by topsoil and planted with grass. While the aesthetics of underground trenches may be better than surface trenches, maintenance can be more difficult and costly (particularly, if the trench must be covered by pavement or concrete). Often "out-of-sight" means "out-of-mind". Consequently, underground trenches should only be installed when strong, enforceable maintenance agreements can be secured from the property owner.

DESIGN 1:

Over-sized Pipe Trench (Figure 5.5). In some designs, an oversized corrugated metal pipe is placed within the trench. Holes are drilled through the pipe to allow runoff to drain to the stone reservoir and then into the subsoil. The oversized pipe is protected from clogging by a layer of filter fabric. The primary advantage of this approach is that it increases the available temporary storage of the trench (i.e., more void space is provided within the pipe than if it was occupied by stone aggregate). The feasibility of the oversized pipe approach is governed by the exfiltration rate of the subsoil, as the pipe must completely drain within 72 hours. As with other underground trench designs, runoff must be pretreated. A two chamber inlet design is shown in Figure 5.5.

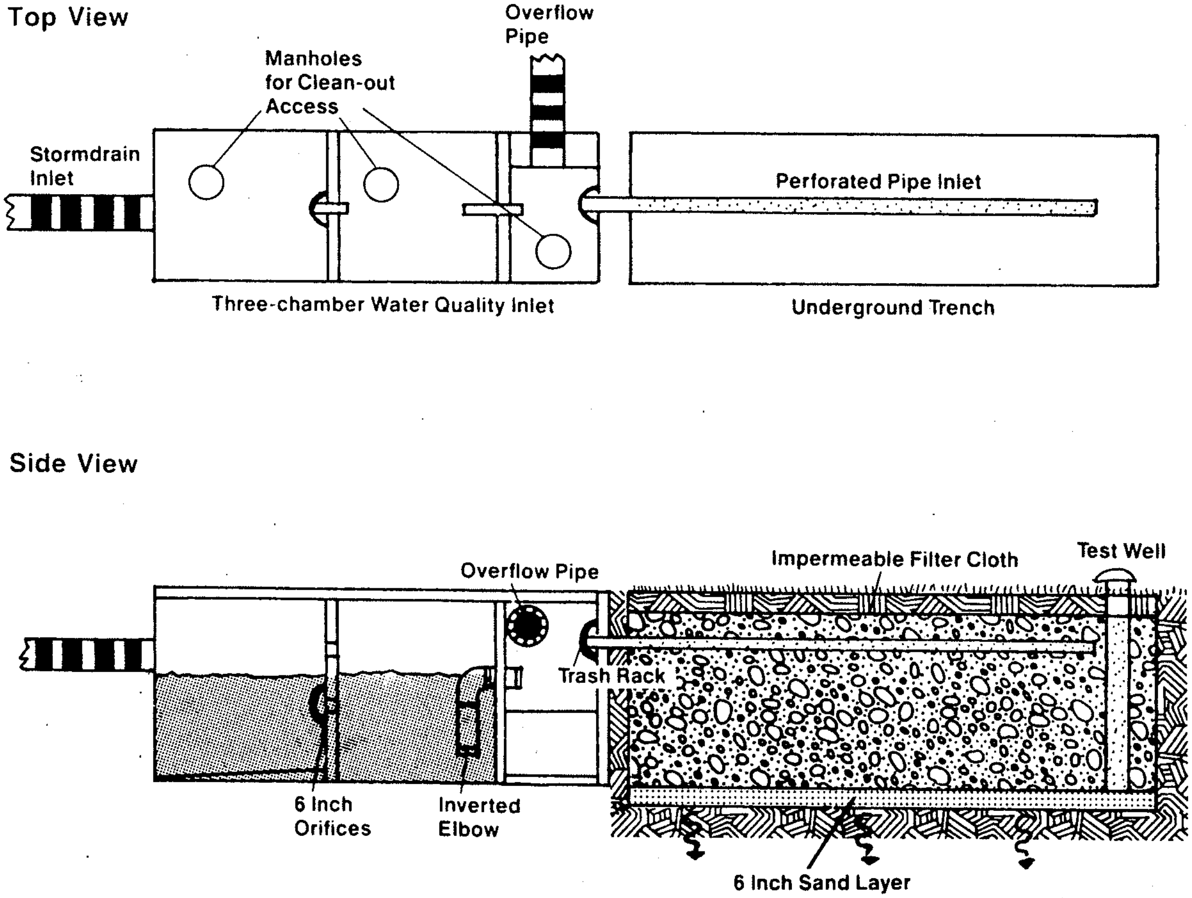
Figure 5.5: Oversized Pipe Trench Design



DESIGN 2:

Underground Trench with Oil/Grit Inlet. (Figure 5.6). Commercial/industrial parking lots produce significant loads of grit and oil, that can, and do, rapidly clog the top of surface trenches, and also provide greater stormwater flows that must be collected by a stormdrain. In these development situations, an oil/grit inlet is needed to pretreat the runoff before it enters the trench. Three-chamber designs (Chapter 8) are popular, whereby the first chamber traps coarse sediment and litter, the second chamber separates out the oil and grease, and the third chamber serves as the inlet to the trench. If the trench is desired for either partial or water quality exfiltration, the third chamber must also have the capability to divert overflow to a storm drain network. More detailed guidance on oil/grit inlets is provided in Chapter 8. A perforated pipe extends along the top of the underground trench so the runoff can be evenly distributed across the stone reservoir.

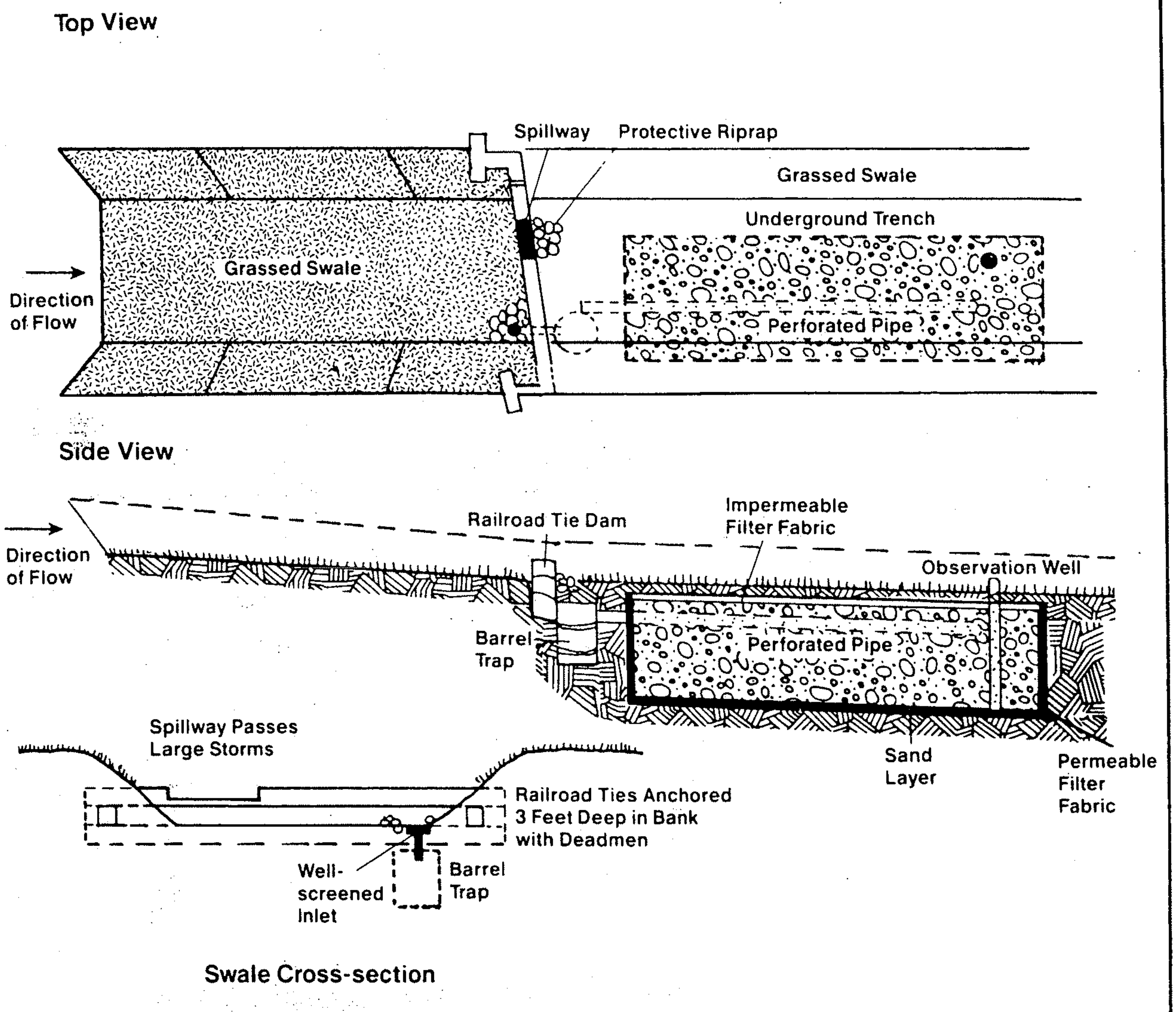
Figure 5.6: Underground Trench with Oil/Grit Chamber



DESIGN 3:

Under-the-Swale Design (Figure 5.7). A surface trench located in a swale may not always be a popular choice for nearby residents. An alternative approach is to place a railroad tie weir across the swale, drop a barrel inlet at the base of the weir to trap sediment, and extend a perforated pipe from the barrel and along the top surface of the trench to distribute runoff evenly. The top of the trench is then covered by at least two layers of nearly impermeable geo-textile, with a 6-12 inch layer of topsoil laid on top. After grass is established, only the test well and railroad tie weir will be visible to residents.

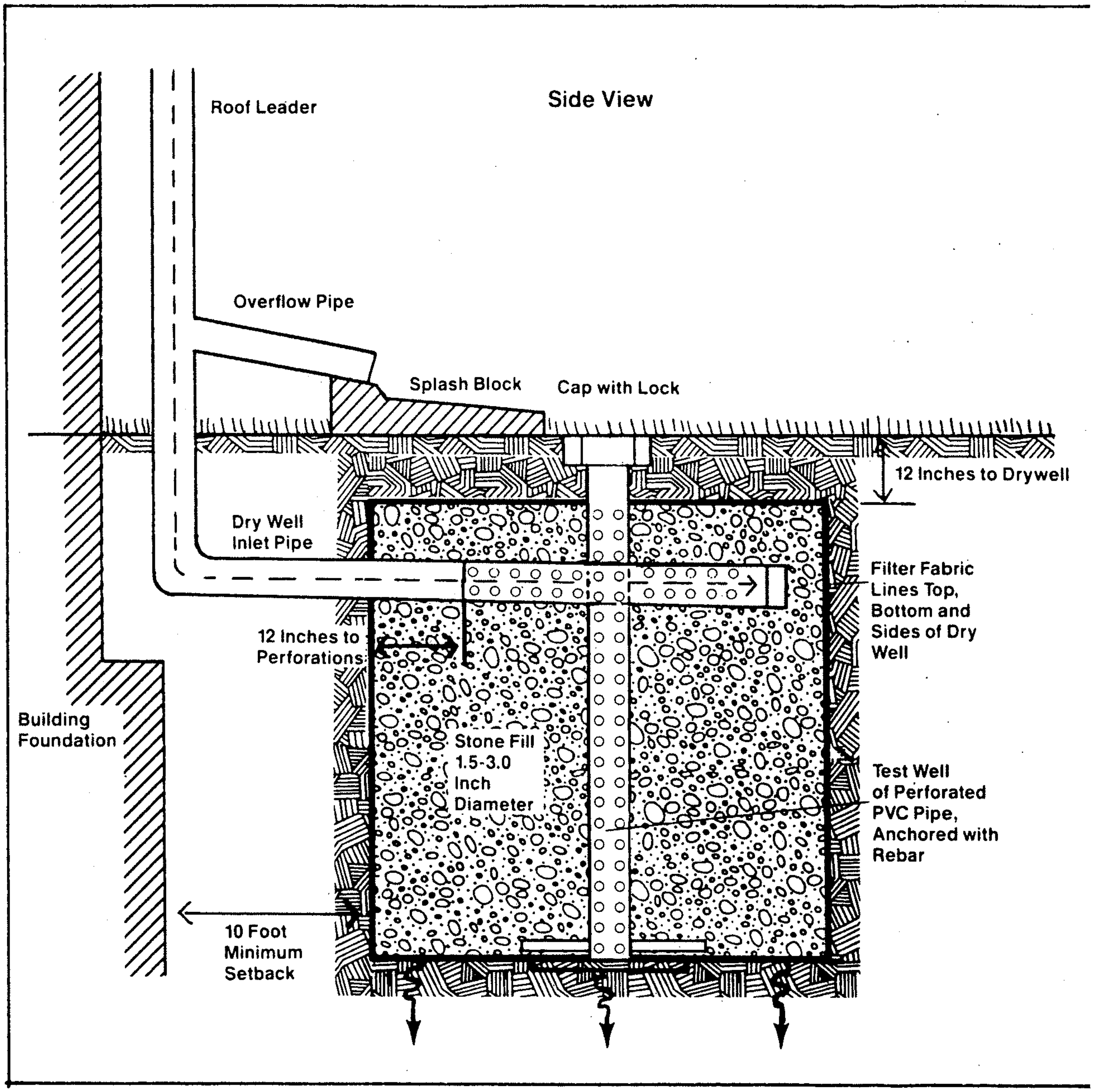
Figure 5.7: Under-the-Swale Trench Design



DESIGN 4:

Dry Well Designs. (Figure 5.8). Dry wells are a basic trench variation which are designed exclusively to accept rooftop runoff from residential or commercial buildings (Figure 5.8). Additional guidance on dry well design is available from Md WRA (1984). Basically, the leader from the roof is extended into an underground trench, which is situated a minimum of ten feet away from the building foundation. Rooftop gutter screens are needed to trap any particles, leaves and other debris, and must be regularly cleared.

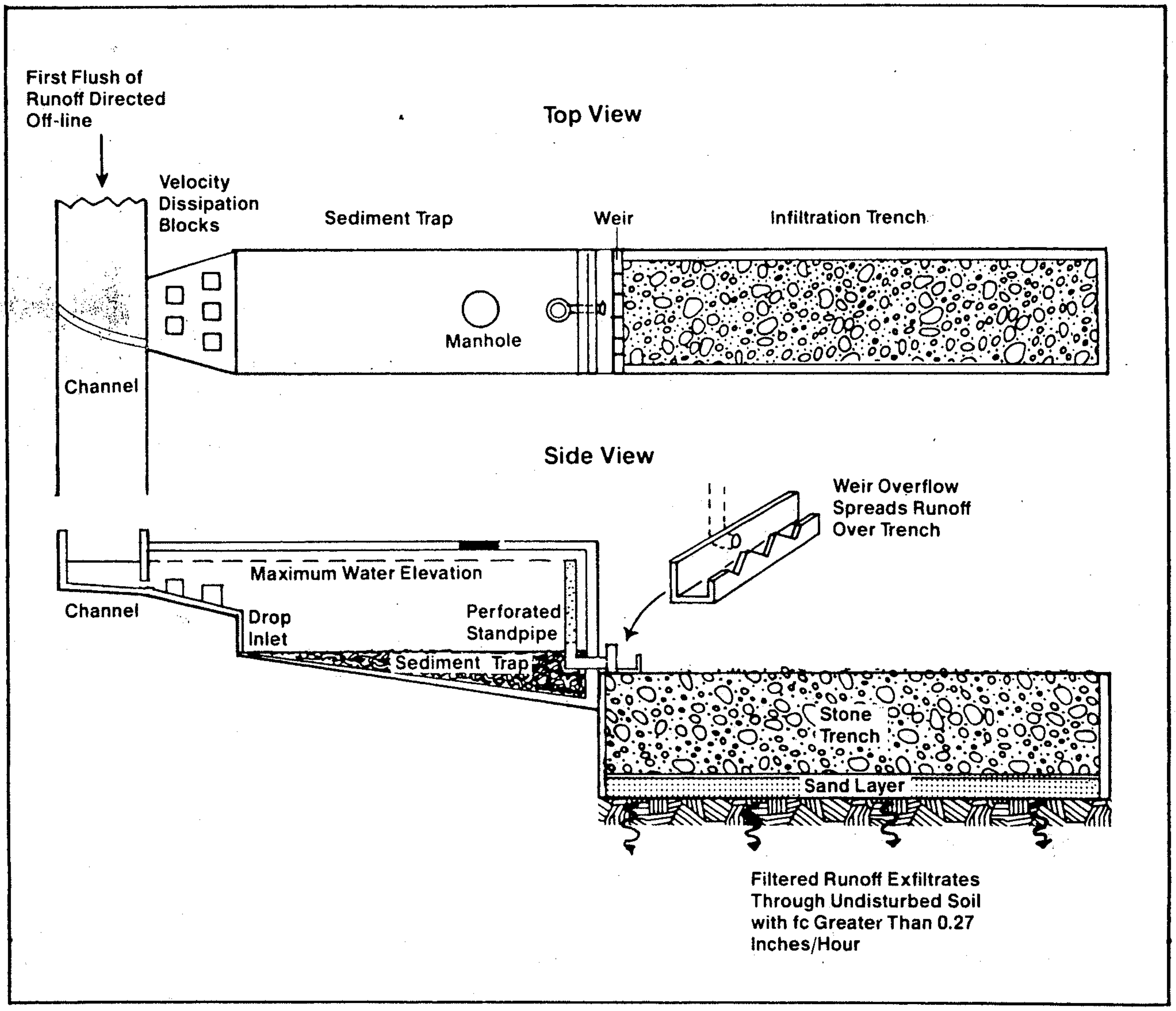
Figure 5.8: Dry Well Design (adapted from Md WRA, 1986)



DESIGN 5:

Off-Line Trench System Designs. (Figure 5.9). Several designs have been originated in Texas (Austin DPW, 1986) that utilize a combination of off-line sediment traps, sand filters and infiltration trenches to treat the first flush of runoff. In one design, a weir is placed across a natural or man-made channel that diverts runoff into an off-line sediment trap. After sediment drops out, the runoff enters a vertical perforated pipe that drains to a level-spreading weir. Runoff then passes over a sand filter to remove any fine particulates or grease remaining in the runoff. After percolating through the sand filter and a layer of permeable filter fabric, runoff is stored in a gravel or stone reservoir, and then exfiltrated into the subsoil (alternatively, runoff from the gravel or stone reservoir can be collected by an underdrain network and be returned to the stream). Some general sizing rules for the area of the sediment trap and trench surface area adapted from Austin DPW (1986) are shown in Figure 5.9.

Figure 5.9: Off-line Trench System Design



EFFECTIVENESS IN STORMWATER CONTROL

Infiltration trenches have the potential to nearly reproduce natural, pre-development hydrologic conditions.

Peak Discharge Control

Full exfiltration trenches completely attenuate the peak discharge associated with the design storm, often well below pre-development levels. For physical and economic reasons, however, it is not always practical to provide enough storage for very large and infrequent design storms (e.g., the 10 or 100 year storms). In such instances, partial exfiltration trenches are used to pass the design storm, and peak discharge control is provided for either the 2 or 10 year design storm. Water quality exfiltration cannot adequately control peak discharges for most design storms, because of their minimal storage capacity, but can reduce the storage needs of a downstream control device in some cases.

Groundwater Recharge

Most trench systems are able to divert a large fraction (60-90%) of the annual runoff volume into the soil. This enhanced recharge helps to maintain flow levels in small headwater streams during critical dry weather periods. Maryland WRA (1986b) estimates that even the smaller sized water quality exfiltration systems can maintain summer baseflow levels to within 90% of natural pre-development conditions. When full exfiltration trenches are utilized, it is actually possible to increase summer baseflow levels slightly above pre-development levels, as diverted runoff is not subject to losses via transpiration.

Volume Control

Unlike retention/detention ponds, trenches can effectively reduce the increase in post-development runoff volume produced during small and moderate sized storms. Storm runoff exfiltrated into the soil profile does not normally appear as part of the downstream storm hydrograph. The effectiveness of a trench in reducing storm runoff volumes is a function of the degree of exfiltration attained (i.e, full exfiltration is better than partial exfiltration, which in turn, is better than water quality exfiltration).

Streambank Erosion Control

The superior hydrologic performance of exfiltration systems should prevent serious streambank erosion immediately below the site. However, since trenches are applied to very small sites, and will only manage stormwater from a small portion of a stream's watershed, other practices (such as extended detention or infiltration basins) must be installed elsewhere in the basin to provide comprehensive protection.

POLLUTANT REMOVAL

As with porous pavement BMPs (see Chapter 7), infiltration trenches are not really intended to provide much removal of coarse particulate pollutants. These must be removed by a pre-treatment device before they enter the trench. Fine particulates and soluble pollutants are effectively removed after exfiltrating through the trench and into the soil. Several decades of experience of land disposal of wastewater has shown that the soil layer is a highly effective and normally safe means of removing pollutants (MWCOG, 1979). Removal mechanisms involve sorption, precipitation, trapping, straining and bacterial degradation or transformation, and are quite complex. The actual removal rates for an individual pollutant depend on its solubility and biochemistry.

Full and Partial Exfiltration Trenches

Table 5.1 provides estimates of pollutant removal rates that might be expected for full exfiltration systems, based on field testing of similar rapid infiltration land treatment systems (NVPDC, 1979; US EPA, 1977).

Table 5.1: Estimated Long-Term Pollutant Removal Rate for Full Exfiltration Trenches

URBAN POLLUTANT	REMOVAL RATE	LIMITING FACTOR
SEDIMENT	99%	Should actually be trapped before reaching the trench.
TOTAL PHOSPHORUS	65-75%	Leaching of remineralized organic P.
TOTAL NITROGEN	60-70%	Leaching of soluble nitrate.
TRACE METALS	95-99%	Behavior similar to sediment.
BOD	90%	Leaching of dissolved organic matter.
BACTERIA	98%	Straining.

Water Quality Trenches

A significant portion of the annual runoff volume will bypass a water quality trench, and is not then subject to removal by exfiltration. Therefore, the pollutant removal capability of water quality trenches are somewhat lower than other designs. As noted earlier, there are two sizing rules for water quality trenches:

SIZING RULE 1: Trench storage volume should be equivalent to 0.5 inches of runoff per impervious acre in the contributing watershed (Md WRA, 1986b).

SIZING RULE 2: Trench storage volume should be capable of storing the runoff produced from a one inch storm over the contributing watershed (1.0*Rv*A) (see Chapter 1 for an explanation of variables).

The comparative runoff capture efficiency for each of these trench sizing rules was evaluated using observed runoff time-series for 300 storms at seven Washington, D.C. NURP sites (imperviousness: 11-27%, "B" soils). A series of calculations were made to determine the runoff volume diverted into, and bypassed over hypothetical trenches according to the water quality rules shown above. The results are shown in Figure 5.10. Under Rule 1 (0.5 inches/impervious acre), approximately 40-50% of storm runoff volumes is captured and exfiltrated over the long-term. For the more generous Rule 2 (runoff from one inch storm), capture efficiencies on the order of 65-75% of storm runoff volumes can be expected.

Actual pollutant removal rates in water quality trenches are slightly higher than the runoff volume capture efficiency, primarily because of the first flush phenomenon (Griffin et al., 1980; Md WRA, 1986b). That is, a greater portion of storm pollutant loads are delivered during the early part of storms due to the rapid wash-off of accumulated pollutants (Sartor and Boyd, 1977). Based on local modeling studies (NVPDC,1979) and field studies (Griffin et al., 1980) of the first flush effect, expected pollutant removal rates for water quality trenches are estimated in Table 5.2.

Figure 5.10: Runoff Capture Efficiency of Water Quality Trenches

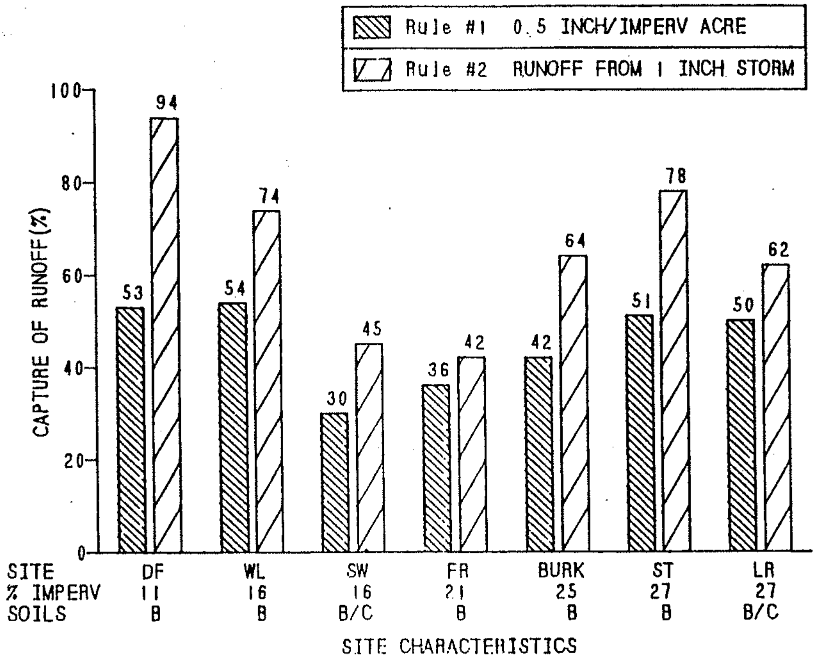


Table 5.2: Estimated Long-Term Pollutant Removal Rate (%) for Water Quality Trenches

POLLUTANT	SIZING RULE 1	SIZING RULE 2
SEDIMENT	75%	90%
TOTAL PHOSPHORUS	50-55%	60-70%
TOTAL NITROGEN	45-55%	55-60%
TRACE METALS	75-80%	85-90%
BOD	70%	80%
BACTERIA	75%	90%

DESIGN TIPS FOR ENHANCING POLLUTANT REMOVAL

Apart from maximizing the amount of runoff exfiltrated, other steps can be taken to promote enhanced pollutant removal.

Surface Area of the Trench Bottom

Pollutant removal in a trench can be enhanced by increasing the surface area of the trench bottom. This is done by adjusting the geometry to make the trench shallow and broad, rather than deep and narrow. More extensive bottom surface area increases exfiltration rates and also provides more area and depth for soil filtering. In addition, broader trench bottoms reduce the risk of clogging at the soil/filter cloth interface by spreading exfiltration over a wider area.

Nature of Subsoils

The greatest sorption of nutrients and metals occurs in soils with a high content of clay and/or organic matter; whereas, the least sorption is observed in sandy soils (US EPA, 1977). The same trend holds true for bacterial densities as well. Unfortunately, soils that maximize sorption and bacterial growth also have low and sometimes unacceptable infiltration rates.

Maximum Draining Time

The trench should be designed to completely drain within three days after the maximum design storm event. Complete drainage is needed to maintain aerobic conditions in the underlying soil long enough to favor bacteria that aid in pollutant removal (and also to ensure that the trench can accept runoff from the next storm).

If a trench is constructed over soils with a marginal infiltration capacity (e.g, loam and silt loam soils), it may be advisable to adjust the depth of the trench so that it drains in two days or less, as a safety margin.

Minimum Draining Time

Moderate to poor pollutant removal has been observed in partial exfiltration systems which hold water less than 6 hours (MWCOG, 1983b). Short residence times do not allow for adequate exfiltration, and thus limit pollutant removal capability. Several design factors lead to rapid passage of runoff through a trench. In particular, perforated underdrains in the bottom of a narrow trench will often be very efficient in collecting runoff, thereby reducing residence time.

Low pollutant removal rates were observed in a perimeter parking trench monitored during the Washington, D.C. NURP project (MWCOG, 1983b). Virtually no exfiltration was observed at the site. The problem was further compounded by low permeability soils and the absence of filter strips on the site. Consequently, the poorly designed trench exhibited no removal of nutrients and soluble trace metals, and, unfortunately, trapped moderate amounts of coarse particulates that apparently clogged the trench (MWCOG, 1983b).

Maintenance

Test wells should be installed in every trench to monitor draining times after installation. The water level in the well should be measured daily after a large storm. If the trench does not completely drain after 3 days, it usually means that the bottom of the trench has clogged and remedial measures need to be taken to improve performance. Conversely, if a partial exfiltration trench empties completely within a day, it means either the collection efficiency of the underdrain is too great, or the bottom of the trench has clogged, or both. Again, remedial measures will have to be taken to ensure adequate pollutant removal.

PHYSICAL SUITABILITY AT THE SITE LEVEL

Before a trench is constructed, the site should be carefully evaluated to be determine whether it is feasible and practical to rely on exfiltration to dispose of runoff. The following factors need to be examined early in the site planning stage to adequately screen the site. More detailed guidance on trench feasibility can be found in Md WRA (1984).

Soils

Trenches are not a feasible option for sites with "D" soils (i.e., infiltration rates of less than 0.27 inches per hour), or any soil with a clay content greater than 30% (as determined from the SCS soil textural triangle). Silt loams and sandy clay loams ("C" soils) provide marginal infiltration rates, and should probably only be considered for partial exfiltration systems (see Table 5.3). Soils with a combined silt/clay percentage greater than 40% by weight are susceptible to frost-heave, and are not good candidates for infiltration trench applications. No matter what soil type is present, the stone subgrade must extend below the frost-line (typically 8-12 inches in the Washington D.C. metropolitan area). Also, trenches are not suitable over fill soils that form an unstable subgrade, and are prone to slope failure.

If the soils at a site pass these tests, a series of soil cores or trenches should be taken at the site, to a depth at least five feet below the anticipated level of the stone reservoir bottom. These should be examined for evidence of any impermeable soil strata that might impede infiltration, such as localized clay lenses, hardpans, or fragipans. The presence of such layers do not necessarily preclude a trench, as long as the stone reservoir completely penetrates them.

Slope

An underground trench is not a feasible option on sites with a slope greater than 20%. Surface trenches are not recommended when contributing slopes are greater than 5%. The slope of the bottom of the trench should be close to zero to evenly distribute exfiltration, unless the design includes a positive outlet.

Depth to Bedrock

At least four feet of clearance will be needed between the bottom of the stone reservoir and the bedrock level. Depth to rock can be estimated from local soil maps but should always be confirmed by several soil test borings.

Depth to Seasonally High Water Table

A minimum of two to four feet of clearance is needed from the bottom of the stone reservoir to the seasonally high water table. This is readily determined by soil borings taken during a wet period.

Proximity of Wells and Foundations

Trenches in commercial and industrial areas should be located at least 100 feet away from a drinking water well to minimize the possibility of groundwater contamination, and should be situated at least 10 feet down-gradient and 100 feet up-gradient from building foundations.

Maximum Depth of Reservoir

To insure that the stone reservoir completely drains in 72 hours, it may be necessary to limit the depth of the stone reservoir when underlying soils have relatively low exfiltration rates. These limits are shown for various soil textures in Table 5.3. If necessary, the dimensions of an infiltration trench would have to be modified in order to accommodate the necessary volume without exceeding the maximum depth limits.

Table 5.3: Soil Limitations For Infiltration Trenches

SOIL TEXTURE	MINIMUM INFILTRATION RATE (fc-inches/hour)	SCS SOIL GROUP	MAXIMUM DEPTH OF TRENCH (in)	
			48 hours	72 hours
Sand	8.27	A	992	1489
Loamy Sand	2.41	A	290	434
Sandy Loam	1.02	B	122	183
Loam	0.52	B	62	93
Silt Loam	0.27	C	32	49
Sandy Clay Loam	0.17	C	20	31
Clay Loam	0.09	D	11	16
Silty Clay Loam	0.06	D	7	11
Sandy Clay	0.05	D	6	9
Silty Clay	0.04	D	6	7
Clay	0.02	D	2	4

Sources: Maryland WRA (1984) and Shaver (1986).

Watershed Size

Md WRA (1984) suggests that trenches be restricted in size to serve drainage areas of less than 5 acres. This guideline reflects the fact that larger watersheds are more practically and cost-effectively served by other BMPs.

Space Limitations

The application of surface trenches could conceivably be space-limited on some "tight" sites because of the 20 foot buffer strip requirement.

INFILTRATION TRENCH COSTS

Predicting Infiltration Trench Costs

A general planning estimate of trench costs can be made using equation 5.1 (Wiegand et al, 1986):

(EQ 5.1) C= 26.6 (Vs**0.63)

where C = construction cost in 1985 dollars, and
Vs = storage volume (cf) of the void space in the trench (≈40% of the excavated trench volume).

The planning equation should not be used if trench storage volumes are greater than 10,000 cubic feet, and does not include costs related to pretreatment of runoff (special inlets or grass filters). An additional 25% should be added to the cost estimate to cover contingency costs.

A more accurate infiltration trench cost estimate can be derived using the in-place unit cost data for infiltration trench components supplied in Table 5.4. Component costs for trenches fall into five general categories, and the quantity of each component can be quickly estimated from trench geometry. The five categories include the following:

1. EXCAVATION constitutes about 20-25% of the total trench cost (Wiegand et al., 1986). Excavation requirements for a trench are equivalent to the total trench volume (width*depth*length).

2. STONE FILL typically comprises 45-55% of the total trench cost. Again, the quantity of stone required can be estimated on the basis of trench volume. Stone fill should be clean/washed material ranging from 1.5 to 3 inches in diameter. Bluestone is generally not recommended. In some cases, washed gravel may be substituted for stone fill.

3. FILTER CLOTH needed to line the sides, bottom and optional top protective layer may contribute approximately 10-15% of the total cost. The quantity of filter cloth needed is approximately equal to:
2(width*depth)+2(length*depth)+2(width*length)+10% overlap allowance.

4. INLET AND OUTLET PIPES needed for underground trenches make up about 10-30% of the total cost of the trench.

5. SODDING rather than hydroseeding should be used for filter strips to ensure that the trench is not contaminated by sediment before grass is established.

EXAMPLE 5-1: UNIT COSTING ESTIMATION FOR A HYPOTHETICAL INFILTRATION TRENCH.

Estimate the costs of constructing a planned partial exfiltration surface trench of the following dimensions: 150 feet long, 6 feet deep, and 6 feet wide.

Step 1. Trench Volume = (l)(w)(d) or (150)(6)(6) = 5400 cubic feet.
(converted to cubic yards, 27 cf=1 cy = 200 cubic yards)

Surface Area= (l)(w) or (150)(6) = 900 square feet.

Step 2. Calculate the component costs:

EXCAVATION:	(150)(6)(6)=5400, 5400/27 = 200 cy	@ $2.82/cy	$564
STONE FILL:	(150)(6)(6)=5400, 5400/27 = 200 cy	@ $22.50/cy	$4500
FILTER CLOTH:	2(6*6)+2(150*6)+2(150*6) = 3672, 10% added (3672)(1.10) = 4039 4039/9 = 449 sy	@ $2.71/sy	$1217
INLET PIPE:	50 feet + 6 feet = 56 feet	@ $10.00/ft	$560
SODDING:	(20)(150) = 3000 square feet	@ $.25/sf	$750
TOTAL COST			$7591

Cost-Effectiveness

Figure 5.8 shows the relationship between construction cost and storage volume provided for 53 BMPs installed in the Washington D.C. metropolitan area (Wiegand et al., 1986). As can be seen, infiltration trenches exclusively serve very small areas (< 10,000 cubic feet storage volume), and, in fact, are the only economical BMP employed in this size range. Extended detention and wet ponds are generally not recommended for small watersheds. Dry ponds, which can be applied on small sites, but have little or no pollutant removal capability, are seldom economically competitive when compared with trenches due to the high fixed costs associated with inlets and risers.

While trenches are the most economical BMP application for small sites, they are probably not the most cost-effective BMP for widespread application in a basin, due to economies of scale. As an example, suppose the total stormwater management storage requirement for a large development is 100,000 cubic feet (cf). The total cost of constructing twenty 5000 cf trenches would be slightly over 100,000 dollars; whereas, the cost of constructing two 50,000 cf extended detention ponds or one 100,000 cf wet pond would be approximately 38,000 and 54,000 dollars, respectively.

Table 5.4: In-Place Unit Costs For Infiltration Trench Construction Components

ITEM	UNITS[1]	AVERAGE IN-PLACE UNIT COST[2]	TYPICAL RANGE
Common Excavation	cy	2.82	2.00-5.00
Clear and Grub	ac	2800.00	1,500-3,500
Seed/Mulch	sy	0.58	0.25-1.00
Rip-Rap	sy	38.00	25.00-55.00
Select Fill	cy	3.97	3.00-5.50
Silt Fence	lf	4.11	2.00-5.00
Gabions	cy	114.00	***
Filter Cloth	sy	2.71	2.00-5.00
PVC Pipe			
6 inch	lf	10.00	8.00-12.00
8 inch	lf	10.50	***
10 inch	lf	15.00	***
Stone Fill (1-2")	cy	22.50	15.00-25.00
Clean Washed Sand	cy	14.00	***
Pea Gravel	cy	7.50	***
Stone Tamping	cy	2.00	***
Observation Well	lf	150.00	25.00-400.00
Sediment Control	lf		1,000-8,000

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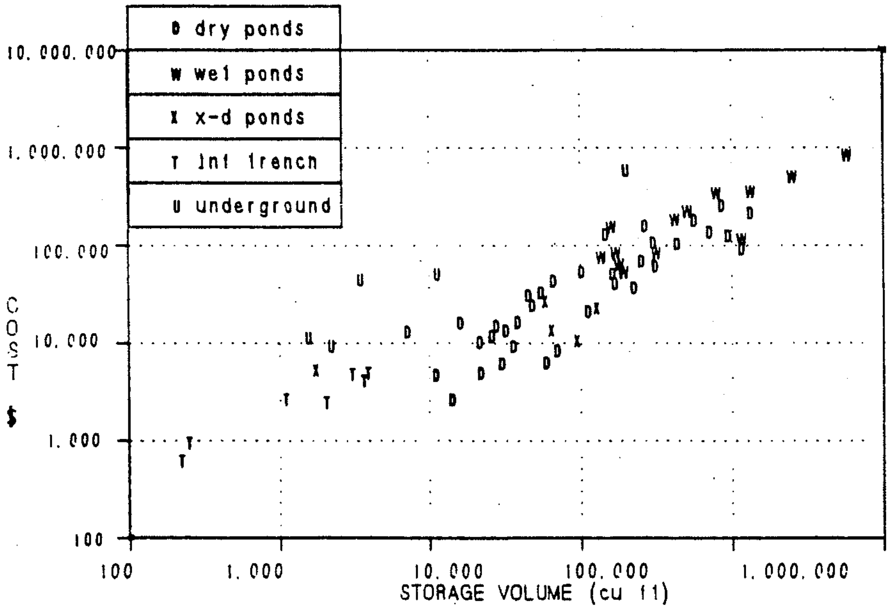
[1] Unit cost data derived from MWCOG (1983a) and supplemented by 45 itemized SWM construction bids or bonding estimates analyzed in the Washington, D.C. area, 1983-1986. Items for which less than five independent estimates were available are denoted by ***. Material costs may vary among jurisdictions and regionally.

[2] cy=cubic yard, sy=square yard, ac=acre, lf=linear foot.

Figure 5.11: Construction Cost Versus Storage Volume in the Washington, D.C. Region



CONSTRUCTION SPECIFICATIONS AND MAINTENANCE REQUIREMENTS

Proper construction and routine maintenance are extremely important for successful trench applications. A substantial number of trenches have failed shortly after being built, primarily due to poor construction practices, inadequate field testing or lack of sediment control. Also, a high percentage of trenches built in the 1970's in the suburban Washington area have failed, primarily because sediment was not filtered or trapped before entering the trench. The discussion below highlights construction and maintenance procedures that should minimize the risk of premature clogging.

Construction Specifications

1. Before the entire development site is graded, the area planned for the trench should be roped off to prevent heavy equipment from compacting the underlying soils.

2. Diversion berms should be placed around the perimeter of the trench during all phases of construction. Sediment and erosion control plans for the site should be oriented to keep sediment and runoff completely away from the trench area. Actual construction of the trench should not begin until after the site is completely stabilized.

3. The trench should be excavated using a backhoe or trencher equipped with tracks or over-sized tires. Normal rubber tires should be avoided since they compact the subsoil and may reduce infiltration capability. For the same reason, the use of bulldozers or front-end loaders should be avoided. Excavated material should be stored at least 10 feet from the trench to avoid backsliding and cave-ins

4. Once the trench is excavated, the bottom and sides of the stone reservoir should be lined with filter fabric to prevent upward piping of underlying soils. The fabric should be placed flush with the sides and bottom with a generous overlap at the seams. Care should be taken in selecting the proper kind of filter fabric, as available brands differ significantly in their permeability and strength. A partial list of approved filter fabric brands is shown in Table 5.5 (Prince Georges County, 1984). If desired, a six inch deep filter of clean, washed sand may be substituted for filter fabric on the bottom of the trench.

5. Clean, washed 1-3 inch stone aggregate should be placed in the excavated reservoir in lifts, and <u>lightly</u> compacted with plate compactors to form the course base. Unwashed stone has enough associated sediment to pose a clear risk of clogging at the soil/filter cloth interface. In some jurisdictions, washed pea-gravel is an acceptable substitute. Where possible, the use of bluestone aggregate should be avoided.

6. A simple observation well should be installed in every trench. Typical details for the well are provided in Figure 5.12. The observation well is needed to monitor the performance of the trench, and is also useful in marking its location. The drain time for a trench can be measured by placing a graduated dip-stick down the well immediately after a storm and again 24 and 48 hours later.

7. Post-construction sediment control is critical. It is therefore important that; 1) sediment and erosion controls be inspected to make

sure they still work, 2) the vegetated buffer strips are established immediately, preferably by sodding, and 3) if hydroseeding is used, reinforced silt fences or Austin triangles must be placed between the buffer and trench to prevent sediment entry before the buffer becomes fully established.

Table 5.5: Approved Geo-Textiles For Use in Infiltration Trenches

Mirafi 140-N

Supac 4NP, 4.5NP, 5NP and 8NP

Typar 3401

AMOCO 4545

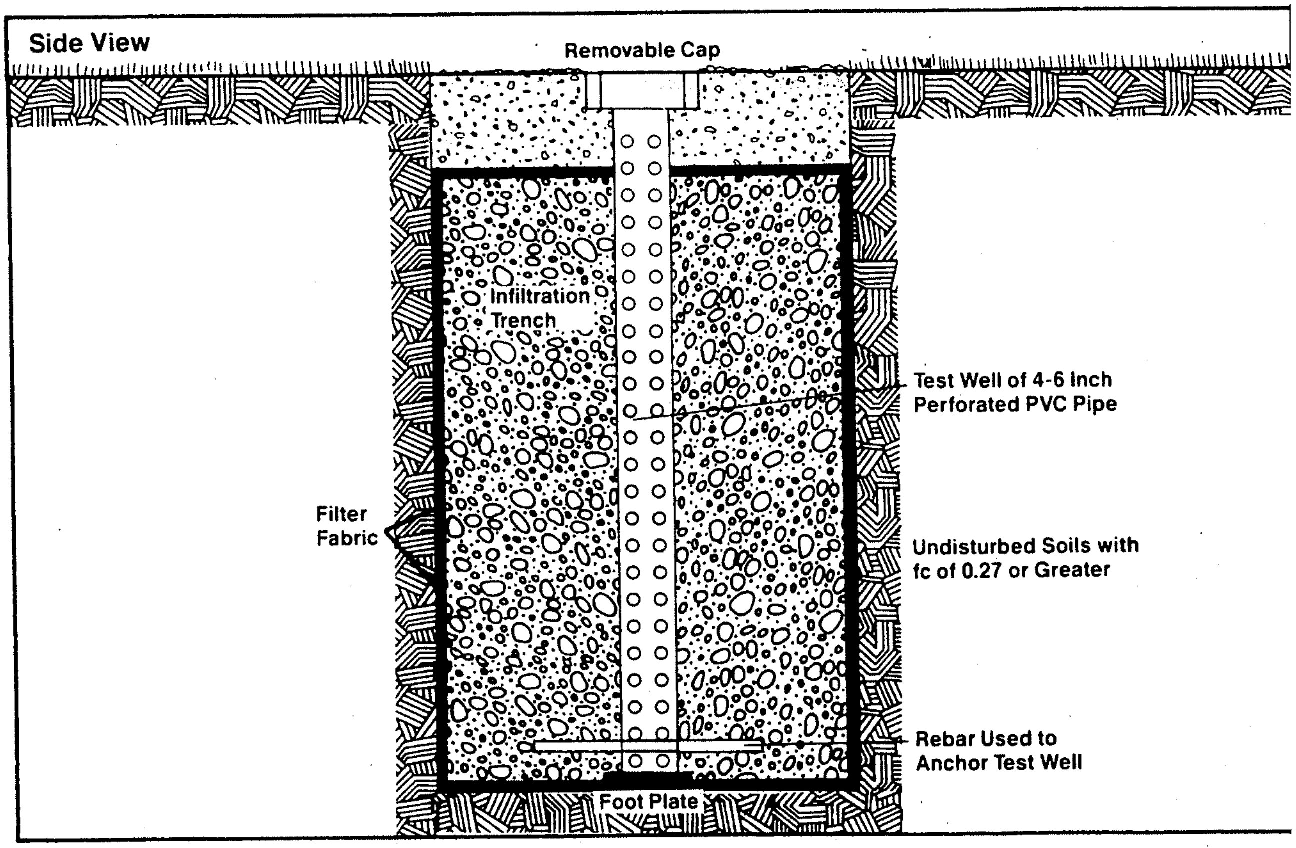
EXXON Geo-textiles No. 125D, 130D and 150D

TerraTex SD

NOTE: This is a partial list of acceptable filter fabrics for use in infiltration trenches available from suppliers in the Washington, D.C. area. The use of a brand name does not constitute an endorsement by MWCOG of any particular product or company.

Source: Prince George's County Stormwater Bulletin No. 4, 1986.

Figure 5.12: Detailed Schematic of a Typical Observation Well



Routine Maintenance

The routine maintenance requirements of trenches are not great. However, getting property owners to actually perform them may be very difficult. Trenches are smaller and more inconspicuous than most other BMPs, and when located underground, may not be visible or accessible. As a result, residents are not likely to exhibit much concern over trench maintenance as they might for more visible BMPs, such as wet or extended detention ponds. For these reasons, a public sector commitment to regularly inspect privately owned trenches is a necessity. Property owners will need to be educated about the function and maintenance requirements of the trench. A legally binding maintenance agreement should be included with the property deed that clearly describes maintenance tasks and schedules. Further, the agreement should grant access for regular inspections, and enable the public sector to perform maintenance (and bill the owners) if the trench has been neglected. Some of the normal maintenance tasks for trenches are detailed below.

INSPECTION

The trench should be inspected several times in the first few months of operation, and then annually thereafter. The inspections should be conducted after large storms to check for surface ponding that might indicate local or wide-spread clogging. Water levels in the observation well should be recorded over several days to check trench drainage. Surface trenches can be inspected by hand by digging with a trowel down to the first layer of filter fabric located one foot below the surface.

BUFFER MAINTENANCE

The condition of the grass buffer strips in surface trenches should be inspected annually. Growth should be vigorous and dense. Bare spots, eroded areas, or "burned out" areas (from road salt or gasoline spills) should be reseeded or re-sodded. Watering and/or fertilization should be provided during the first few months after the strip is established, and may periodically be needed in times of drought.

MOWING

Grass filter strips should be mowed at least twice a year to prevent woody growth as well as for aesthetic reasons. Filter strips in residential areas will need to be mowed more frequently (10 to 14 times per year). Filter strip performance will be impaired if the grass is cut too short (Tollner, 1976). To prevent lawn clippings from clogging the trench, mowers should be equipped with baggers or at a minimum be directed away from the trench.

SEDIMENT REMOVAL

The pre-treatment inlets of underground trenches should be checked periodically and cleaned out when sediment depletes more than 10% of available capacity. This can be done manually or by a vacuum pump. Inlet and outlet pipes should be checked for clogging and vandalism.

TREE PRUNING

Adjacent trees may need to be trimmed if their drip-line (i.e. the reach of the branches) extends over a surface trench so that tree leaves do not clog the trench. In addition, pioneer trees that start to grow in the vicinity of a trench should be removed immediately thereby avoiding root puncture of the filter fabric through which sediment might enter the structure.

Non-Routine Maintenance

The primary non-routine maintenance task involves rehabilitation of the trench after it becomes clogged. Unfortunately, acceptably designed trenches have only recently come into use in the Washington, D.C. area. As a result, there is no reliable estimate as to how long trenches will function before they clog. Emphasis throughout this chapter has been on designs and procedures which minimize the likelihood of clogging. However, it is probable that some trenches will eventually clog despite careful design, construction and maintenance. Md WRA (1985b) suggests that the longevity of trenches may be on order of 10-15 years.

Clogging in surface trenches is most likely to occur near the top of the trench, between the upper layer of stone and the protective layer of filter fabric. Surface clogging can be relieved by carefully removing the top layer of stone, removing the clogged filter fabric, installing new filter fabric, and cleaning or replacing the top stone layer. The costs for rehabilitating a surface trench are not known, but are not likely to exceed 20% of the initial construction cost.

Clogging of underground trenches is a much more serious problem, as it is likely to occur at the bottom of the trench, at the filter fabric/soil interface. Rehabilitation of an underground trench requires the removal of 1) the topsoil/vegetation layer, 2) the protective plastic layer, 3) entire stone aggregate layer, and 4) the bottom filter fabric layer. Then, the subsoil layer must be tilled to promote better infiltration, and each layer must be replaced. If pavement or concrete are used for the surface layer (instead of topsoil/grass), the rehabilitation effort becomes more difficult and costly.

Total Maintenance Costs

No reliable data is presently available to assess maintenance costs for trenches. Routine maintenance costs will probably run higher for surface trenches than underground trenches, primarily due to the mowing operation needed for the filter strip. As noted above, the opposite is probably true for non-routine maintenance tasks. It is probably reasonable to assume that the cost of rehabilitating an underground trench will be roughly equivalent to the initial construction cost. Surface trench rehabilitation should only be approximately 20% of the initial construction cost; however, there are reasons to expect that the clogging of surface trenches may occur more frequently.

If it is assumed that surface and underground trenches will need rehabilitation every 5 to 15 years, respectively, then an annual maintenance set-aside of 5-10% (surface trenches) and 10-15% (underground trenches) of the initial construction cost may be needed to cover routine/non-routine maintenance expenditures. It must be emphasized that these estimates are highly uncertain. Until more local experience is obtained, the issue of trench maintenance costs remains largely speculative.

ENVIRONMENTAL ATTRIBUTES OF TRENCHES

Impacts to the Natural Environment

Infiltration trenches generally receive high marks for protecting downstream aquatic life, as they maintain the pre-development water balance at the site, minimize streambank erosion and filter out pollutants.

One potential negative impact of trenches is the risk of groundwater contamination. Long-term studies of pollutant migration in soils underneath various infiltration practices indicate only limited downward migration of pollutants through the soil (Nightingale,1987; OWML, 1983; US EPA, 1983). Possible exceptions include very soluble pollutants such as nitrate, chlorides and gasoline. A more definitive assessment of the possible risks of groundwater contamination by trenches is the focus of a current monitoring survey being conducted in Maryland by the U.S. Geological Survey

Impacts on the Human Environment

Trenches should be designed to be an unobtrusive feature of the landscape. Since trenches are largely or entirely underground, they are not likely to have any strongly positive or negative impacts on the human environment.

RELEVANT DESIGN GUIDANCE

A summary of important design features for infiltration trenches can be found in Table 5.6, and are shown in schematic form in Figure 5.10. The following references should also be consulted for more detailed guidance on the design and installation of infiltration trenches:

Maryland Water Resources Administration, 1984. Standards and Specifications for Infiltration Practices.

Maryland Water Resources Administration, 1986. Inspectors Guideline Manual for Stormwater Management Infiltration Practices.

DESIGN SUMMARY: INFILTRATION TRENCHES

- **SITE EVALUATION:**
  Soils must be tested prior to design to determine whether infiltration is feasible for a site. Soil borings should be taken to a depth at least five feet below the anticipated bottom of the trench to check for soil infiltration capability, depth to seasonally high water table, and bedrock level. The minimum field infiltration rate (fc) of the underlying soils should be greater than 0.27 inches/hour.

- **WATERSHED SIZE:**
  The watershed area contributing to each trench should not exceed 5 acres.

- **DEGREE OF EXFILTRATION:**
  To achieve significant pollutant removal, at least one-half inch of runoff per contributing impervious acre should be exfiltrated into the underlying soils. A more efficient design will accommodate the runoff produced from a 1 inch storm over the contributing watershed.

- **CONSTRUCTION:**
  All trenches should be excavated using light equipment, taking care not to compact the underlying soils used for exfiltration. The sides of the trench should be lined with filter fabric to prevent the entry of sediment into the trench. A six inch layer of sand, or filter fabric should be used to line the bottom of the trench. Clean, washed stone aggregate, 1.5-3.0 inches in diameter, should be used for fill, although washed pea-gravel may be an acceptable alternate in some cases.

- **PRETREATMENT OF RUNOFF:**
  To prevent premature clogging of trenches, sediment, grit, and oil must to be removed by a pre-treatment facility before they enter a trench. For surface trenches, a minimum 20 foot wide grass buffer is required as a filter. In addition, a layer of filter fabric placed one foot below the surface of the trench can be used to trap sediments that get through the grass buffer. Pretreatment technologies for Underground trenches include barrel inlets, water quality inlets, and modified catch-basins.

- **MAXIMUM DRAINING TIME:**
  All trenches should be designed to completely drain within 72 hours after the design exfiltration event. This enables the underlying soils to dry out (improving pollutant removal capability) and frees up storage capacity for the next storm. On sites with soils of marginal infiltration capacity (silt loams, loams), it may be advisable to design trenches to drain within 48 hours. This is done by maximizing the surface area of the trench floor, or reducing the depth of the trench, or both.

- **MINIMUM DRAINING TIME:**
  Partial exfiltration trenches should be designed so that all available storage space in the trench is filled before runoff is collected by the underdrain and routed out of the facility. This can be done by installing a perforated pipe (with holes drilled through the bottom) near the top of the trench to collect excess runoff. Perforated underdrains

situated at the bottom of the trench may become too efficient at collecting runoff, and thus reduce pollutant removal.

- OBSERVATION WELLS:
  An observation well, consisting of a well-anchored vertical perforated PVC pipe, should be installed in every trench to monitor its performance. The well should be checked several times within the first few months after construction, by recording trench water depth at 0, 24 and 48 hours after a storm. The clearance rate of runoff (inches/hour) in the trench can be calculated by dividing the drop in water level (inches) by the time elapsed from the end of the storm. A measurement of trench clearance rate should be taken during each annual maintenance inspection. A series of such measurements over the years provides an excellent means of tracking any clogging within the trench.

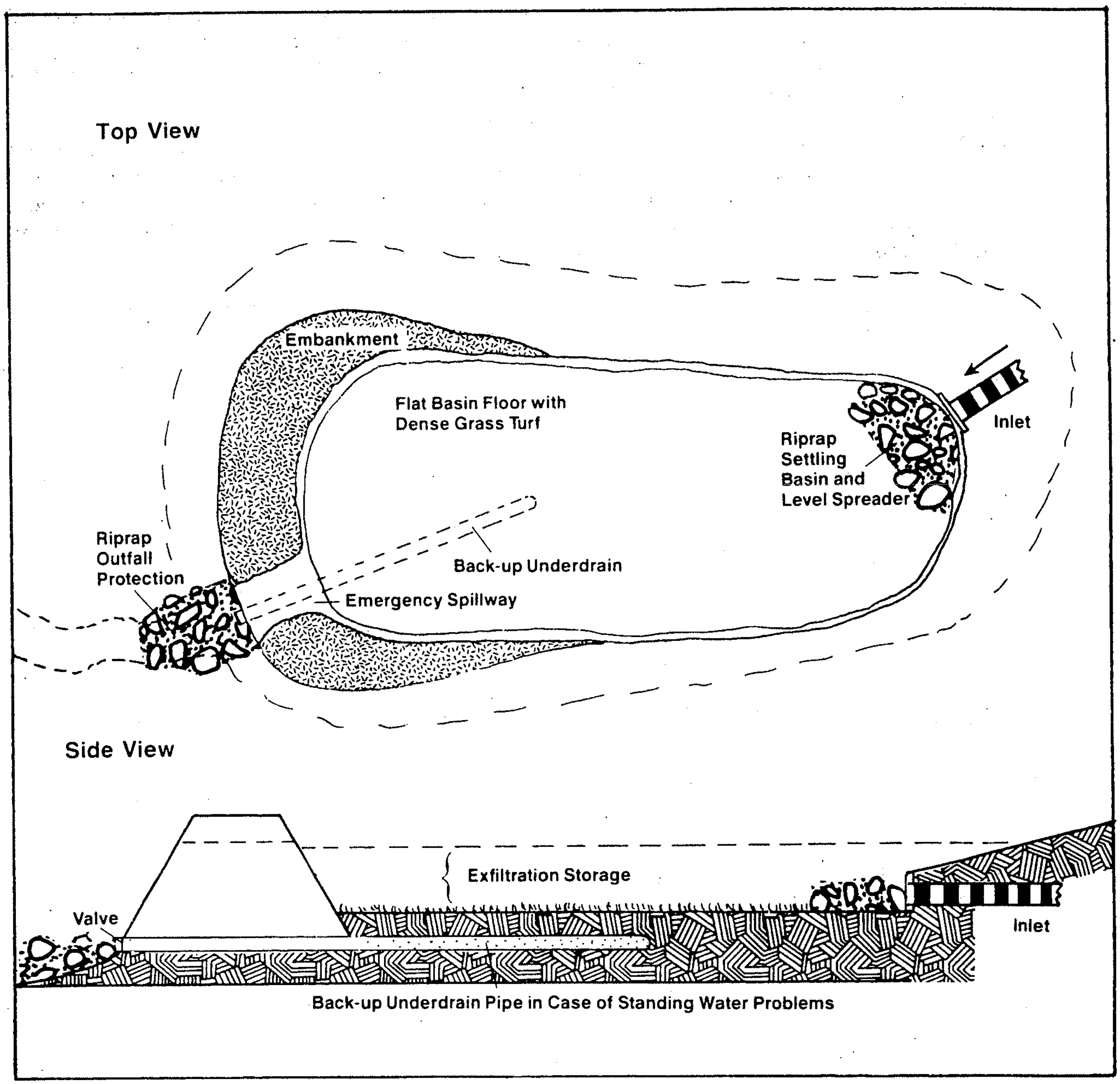
- EROSION CONTROL:
  Trenches should not be constructed until the entire upland contributing area has been stabilized (i.e., after construction is completed). The planned area for the trench should be roped off to prevent compaction by heavy equipment. During construction, sediment and erosion controls such as diversion berms, for example, should be used to keep sediment and runoff completely away from the trench site.

- ANNUAL MAINTENANCE:
  A legally enforceable and binding maintenance agreement should be included in the property deed for each trench that clearly spells out maintenance tasks and schedules. Annual public sector inspections should be conducted to check on the performance of the trench and the required maintenance tasks. These include maintaining a dense grass buffer strip for surface trenches, removing accumulated sediments within the pre-treatment devices of underground trenches, and partially or totally reconstructing the trench in the event of clogging.

CHAPTER 6: INFILTRATION BASINS

Infiltration basins are effective in removing both soluble and fine particulate pollutants borne in urban runoff. Coarse-grained pollutants should generally be removed before they enter a basin. Unlike other infiltration systems, basins can be easily adapted to provide full control of peak discharges for large design storms. Also, basins can serve relatively large drainage areas (up to 50 acres). Depending on the degree of storage/exfiltration achieved in the basin, significant groundwater recharge, low flow augmentation and localized streambank erosion control can be achieved.

Figure 6.1: Schematic of an Infiltration Basin



Basins are a feasible option where soils are permeable and the water table and bedrock are situated well below the soil surface. Both the construction costs and maintenance requirements for basins are similar to those for conventional dry ponds. Infiltration basins do need to be inspected regularly to check for standing water. Experience to date has indicated that infiltration basins have one of the higher failure rates of any BMP.

Advantages of infiltration basins are that they preserve the natural water balance of the site, can serve larger developments, can be used as sediment basins during the construction phase, and are reasonably cost-effective in comparison with other BMPs. Disadvantages of infiltration basins include a fairly high rate of failure due to unsuitable soils, the need for frequent maintenance, possible nuisances (e.g., odors, mosquitos, soggy ground), and some practical design problems.

INFILTRATION BASIN DESIGNS

A schematic of an infiltration basin is shown in Figure 6.1. The appearance and construction of infiltration basins is similar in many respects to conventional dry ponds. An impoundment is formed by excavation or by constructing an embankment. The impoundment stores a defined quantity of runoff, allowing it to slowly exfiltrate through the permeable soils of the basin floor. The floor is graded as flat as possible and a dense turf of grass is established to promote infiltration and bind up deposited sediments. Additional storage can be provided in the basin for temporary detention of the larger runoff volumes associated with the two year and/or ten year design storm, utilizing a conventional riser. An emergency spillway is used to pass runoff volumes in excess of the design storm controlled.

While simple in concept, infiltration basins do present some practical problems from a design standpoint. Problems emerge because infiltration methods are not very good at handling the concentrated flows and sediment loads that are generated from larger watersheds. Thus, basin design must incorporate measures that:

1. Trap excess loads of coarse grained sediment before they enter the basin and clog the surface soil pores on the basin floor.

2. Route design stormflows through the basin without scouring or eroding the basin floor.

3. Route baseflow (if any exists) rapidly through the basin to prevent ponding or standing water.

4. Distribute storm runoff volume evenly over the floor of the basin to maximize exfiltration rates.

5. Provide a back-up drainage system should the infiltration capacity of the basin fail.

Some variations in infiltration basin design that address these problems are discussed below.

Full Infiltration Basin

This simple design is commonly used on sites with extremely permeable soils. The basin is sized to accommodate the entire runoff volume associated with the two year design storm, and the only outlet from the pond is an emergency spillway which passes larger storm events (Figure 6.1). A riprap apron is needed near the inlet to reduce incoming runoff velocities to promote more uniform infiltration. Otherwise, this rudimentary design has no other features for routing stormflow or baseflow through the structure. Consequently, the the use of a full infiltration basin is generally restricted to smaller watersheds (5 to 20 acres) that do not have concentrated, erosive flows.

Combined Infiltration/Detention Basin

This design is one of the more common infiltration basin designs in use today (Figure 6.2). Runoff entering the top of the basin is first trapped in a modified riprap settling basin. Coarse sediment drops out, and the remaining runoff filters through the riprap apron and is spread out over the level basin floor. The depth of runoff in the basin is controlled by a vertical riser. The 2 year control orifice is placed several feet above the bottom of the pond, creating a zone of dead storage. The runoff within the dead storage zone will be completely exfiltrated. If any baseflow exists, a low flow channel should be installed to pass it rapidly through the basin.

Figure 6.2: Combined Infiltration/Detention Basin Design

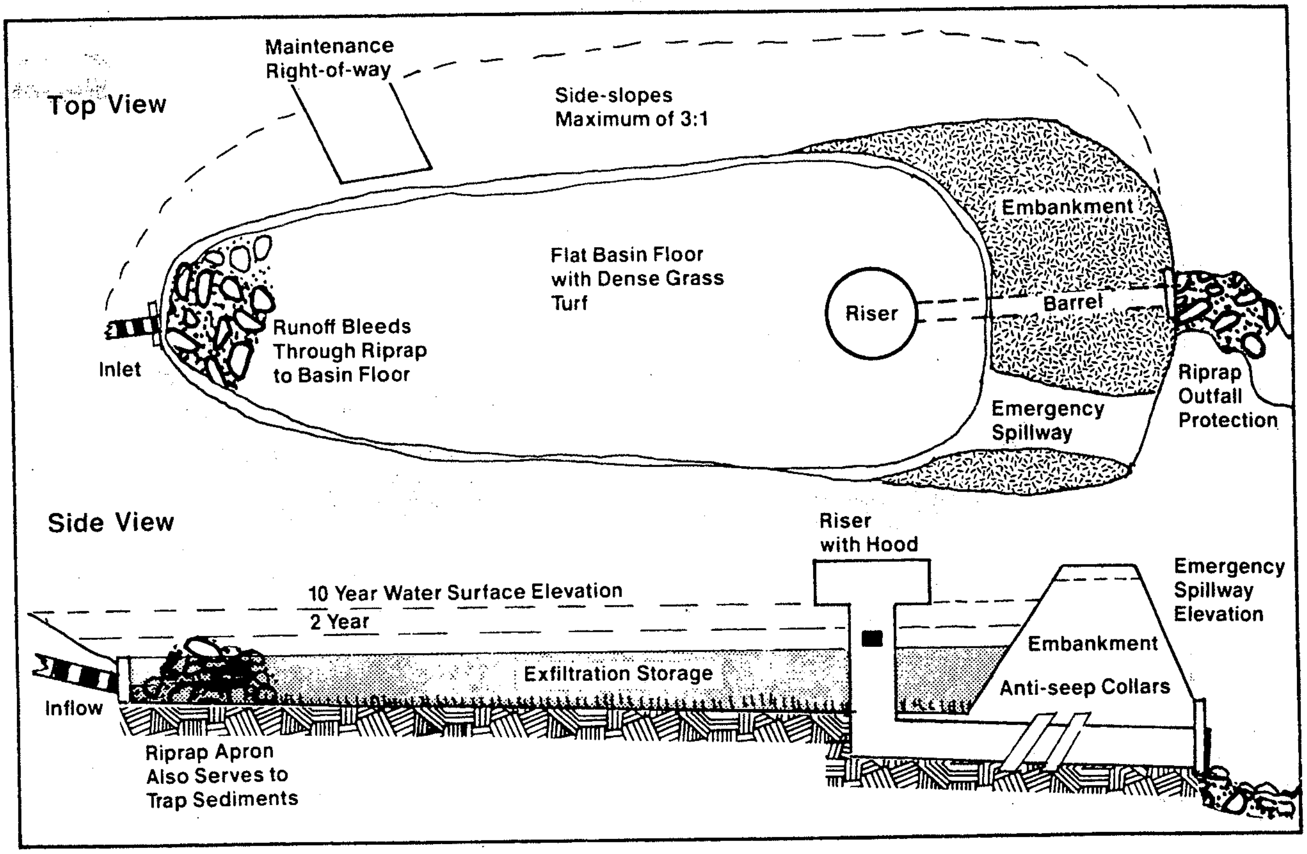
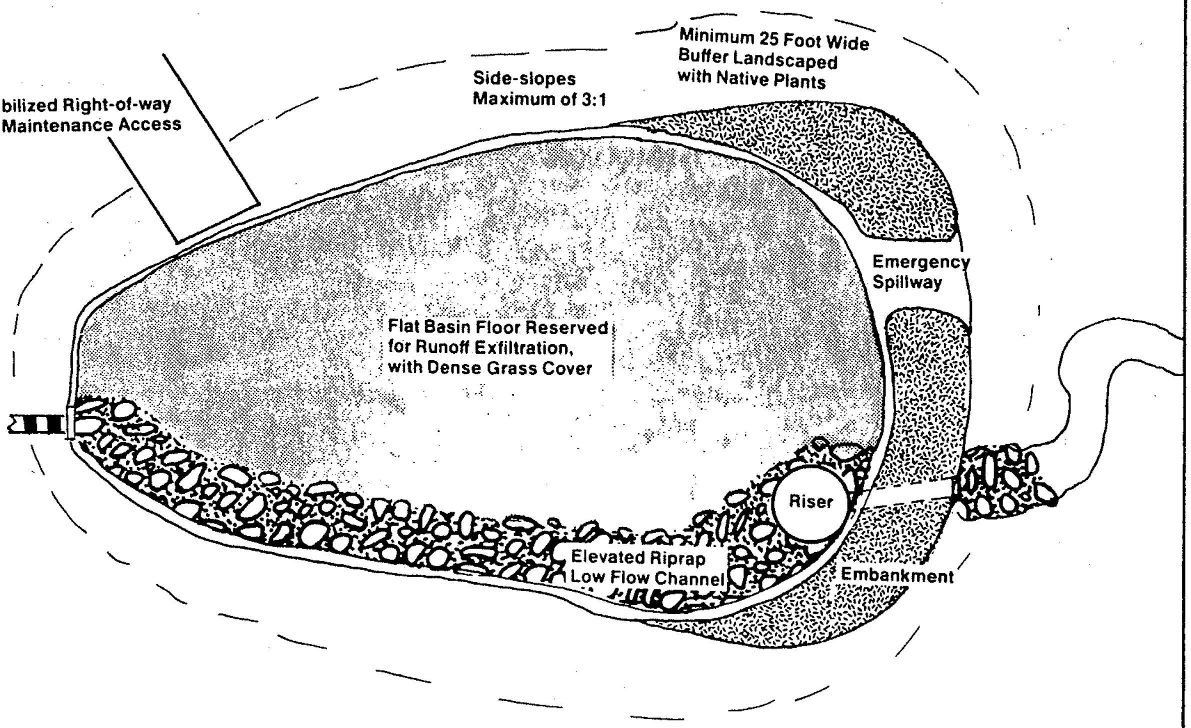
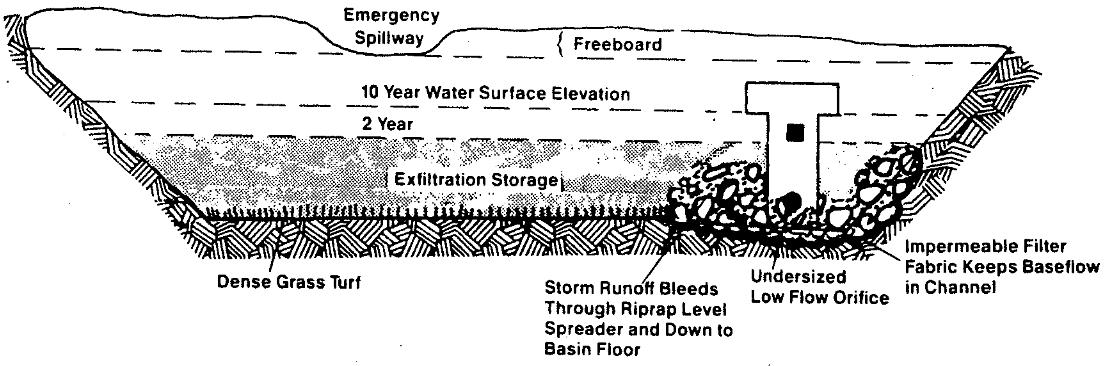


Figure 6.3: Side-by-Side Infiltration Basin Design

Top View



Cross-sectional View of Embankment



Runoff volume in excess of the dead storage volume drains through the low flow orifice, while the very large runoff volumes associated with the design storm spills over the drop inlet at the top of the riser. Extremely large storms (such as the 10 or 100 year storm) are routed through the basin and discharged via the emergency spillway. If the basin is located over soils with marginal infiltration capacity, it may be prudent to extend some capped underground perforated pipes from the riser to drain the basin floor in the event that exfiltration rates are overestimated. The pipes can then be uncapped later, if it is found that the basin suffers from chronic standing water problems or local groundwater mounding.

This basic design can be applied to serve most residential and commercial developments. However, it must be modified if the basin is expected to receive a sustained input of baseflow, or large sediment loads.

Side-by-Side Basin

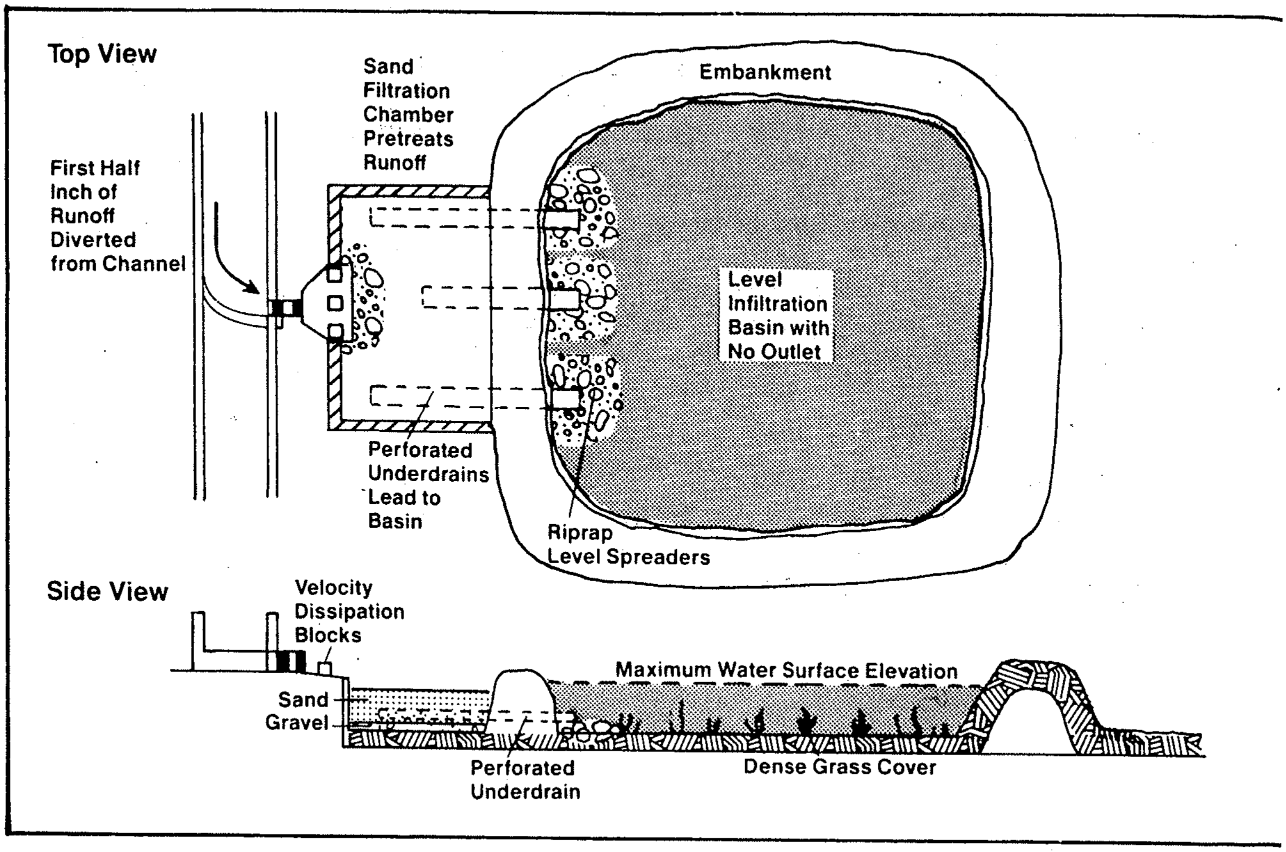
The design of larger infiltration basins must address the tricky problem of routing small baseflow and large stormflows through the basin, while still providing good exfiltration capability for small and moderate sized storm events. One solution is a side-by-side design (Figure 6.3), wherein a riprap pilot channel is constructed along one margin of the basin and extends all the way to the riser. The pilot channel is elevated several feet above the basin floor. Baseflow is confined to the pilot channel (by a layer of impermeable geo-textile) and travels directly to an undersized low flow orifice at the base of the riser, and then out of the basin.

Stormflow pulses are also directed through the pilot channel. However, once incoming stormflows reach a given depth they are no longer confined by the impermeable geo-textile, and may leak through the riprap and down across the basin floor. Storm runoff that does travel all the way to the riser is then diverted down a riprap bench and back into the basin floor. The invert of the low flow orifice is set to form a dead storage zone down to the basin floor that stores the equivalent of the first flush runoff volume.

Off-line Infiltration Basins

Off-line designs are used to divert and exfiltrate the first flush runoff volume from a storm sewer or surface channel. They are particularly useful in development situations where exfiltration cannot be achieved by a downstream stormwater detention facility due to soil limitations. An off-line design modified from Austin DPW (1986) is shown in Figure 6.4. This design utilizes a combination of an off-line sand filter and infiltration basin to treat the first flush runoff volume. A weir is placed across a natural or man made channel that diverts runoff into an off-line sand filter. After percolating through the sand filter, runoff is collected by underdrains which lead to a level, vegetated infiltration basin. This is a particularly appropriate design for sites that drain land uses which produce high sediment or hydrocarbon loads.

Figure 6.4: Off-line Infiltration Basin Design



EFFECTIVENESS IN STORMWATER CONTROL

Infiltration basins are unique among all the BMPs reviewed here in that they can most closely reproduce natural, pre-development hydrologic conditions. When properly designed and sized, infiltration basins can completely manage peak discharges from design storms, provide groundwater recharge and low flow augmentation, reduce storm runoff volumes, and protect downstream channels from erosion.

Peak Discharge Control

Full infiltration basins are typically sized to store and exfiltrate the entire runoff volume associated with the design storm. These basins often have no direct outlet apart from an emergency spillway which passes extreme storm events. As a result, full infiltration basins will control post-development peak discharge rates at or below pre-development levels.

The infiltration/detention basin design can completely attenuate the peak discharges associated with the design storm to pre-development levels. As with other detention ponds, the optimum level of flood control is achieved when multiple design storms are controlled (in particular, the 2 and 10 year storm events). The dead storage provided for exfiltration purposes also helps to partially or completely attenuate the peak discharges of intermediate design storms.

Off-line infiltration basin systems are not intended to provide peak discharge control for the design storm. However, these smaller basins often reduce the size of downstream stormwater facilities.

Groundwater Recharge

Infiltration basins divert a significant fraction of the annual runoff volume back into the soil. This enhanced recharge can maintain flow levels in small headwater streams during critical dry weather periods. Without the artificial recharge, even moderate levels of development will drastically reduce summer flows in small and mid-sized streams, that in turn, severely stress aquatic life and degrade water quality. Md WRA (1986b) suggests that infiltration systems sized according to Sizing Rule 1 or 2 will be able to maintain summer baseflow levels to within 90% of natural, pre-development conditions.

Volume Control

Infiltration basins effectively reduce the increase in post-development runoff volume produced from small and moderate sized storms, as all of the runoff volume retained will be temporarily stored and then gradually exfiltrated in the dead storage area. Full infiltration basins are capable of providing almost complete volume control.

Downstream Effects

Random siting of infiltration/detention basins in a watershed may not result in the desired downstream reduction in peak discharges for the design storm, because of differences in the location and timing of releases from individual basins. For example, a basin situated at the bottom of a watershed may detain stormwaters just long enough to coincide with the arrival of the upstream flood peak, and thus add to the cumulative watershed peak discharge. Therefore, it is advisable to perform detailed hydrological modelling to assess the hydrological impacts of individual basins on the cumulative watershed hydrograph; and to locate basins or adjust release rates accordingly. Random siting will normally not be a problem with full infiltration basins.

Streambank Erosion Control

Infiltration basins can help to control both the magnitude and frequency of post-development bankfull discharges. The dead storage provided for exfiltration in a basin functions in the same manner as the storage/release of an extended detention pond (i.e., a defined runoff volume is retained and slowly released over time). Therefore, it is reasonable to assume that infiltration basins can exert roughly the same degree of control over post-development increases in bankfull flood frequency as extended detention ponds, when similar runoff volumes are stored. As noted earlier, the degree of control of bankfull flooding frequency is a function of the volume of runoff detained/exfiltrated (Chapter 3). Control of the runoff volume generated by a 1.0 to 1.5 inch storm should reduce the frequency of bankfull flooding to pre-development levels, and thus keep downstream channels relatively stable. The smaller runoff volumes specified for water quality control (0.5 inch/impervious acre) can also curtail the number of bankfull flooding episodes sharply, but probably not to pre-development levels.

POLLUTANT REMOVAL

The pollutant removal capability of infiltration basins has not been extensively tested in the field. As with other infiltration systems, pollutant removal is achieved by diverting stormwater runoff through the floor of the basin and into the soil. Limited field data from other infiltration systems, and several decades of experience in land disposal of wastewater suggest that the soil is a highly effective and normally safe filter for removing pollutants (MWCOG, 1979). Removal mechanisms involve sorption, precipitation, trapping, straining and bacterial degradation or transformation. Removal mechanisms are quite complex, and actual removal rates depend on the solubility and chemistry of each individual pollutant. Table 6.1 provides estimates of pollutant removal rates that might be achieved in various sized infiltration basins. The differences in estimated removal rates shown reflect differences in the amount of the annual runoff volume that passes through the basin without exfiltrating. Three frequently used basin sizing rules are defined below.

SIZING RULE 1: Basin sized to store and exfiltrate 0.5 inches of runoff per impervious acre in the contributing watershed (Md WRA, 1986b), with excess runoff only temporarily detained.

SIZING RULE 2: Basin sized to store and exfiltrate the runoff produced from a one inch storm over the contributing watershed (1.0*Rv*A) (see page 1.10), with excess runoff only temporarily detained.

SIZING RULE 3: Basin sized to store and exfiltrate runoff volumes up to and including the two year design storm runoff volume, with excess runoff volume associated with larger storms only briefly detained or bypassed entirely.

Table 6.1: Estimated Long-Term Pollutant Removal Rates (%) For Infiltration Basins

POLLUTANT	SIZING RULE 1 0.5 in/imperv acre	SIZING RULE 2 1.0 inch*Rv*A	SIZING RULE 3 2 yr runoff volume
SEDIMENT	75%	90%	99%
TOTAL PHOSPHORUS	50-55%	60-70%	65-75%
TOTAL NITROGEN	45-55%	55-60%	60-70%
TRACE METALS	75-80%	85-90%	95-99%
BOD	70%	80%	90%
BACTERIA	75%	90%	98%

NOTE: Estimated removal efficiencies based on runoff capture efficiency of exfiltration storage (see Chapter 5, page 5.13 for derivation).

DESIGN TIPS FOR ENHANCING POLLUTANT REMOVAL

The estimated removal rates shown in Table 6.1 presume ideal exfiltration conditions within the basin. The following design steps can help to assure that these conditions will occur in the field.

Surface Area of Basin Floor

The rate and quantity of exfiltration is enhanced by increasing the surface area of the basin floor, particularly when the soil infiltration capacity is marginal. Thus, large, relatively shallow basins are preferable to those which are small and deep. Excess surface area in the basin floor can also compensate for diminished infiltration capacity resulting from surface clogging.

Tilling

If heavy equipment is used to grade the basin floor, the floor should be immediately filled to offset any compaction that has taken place.

Reducing Incoming Water Velocities

Inlet channels leading to the basin should be stabilized to prevent incoming runoff velocities from reaching erosive levels and scouring the basin floor. This is customarily done by riprapping the inlet channels or pipe outfalls. A second design objective of the riprapping is to spread the incoming runoff more evenly over the surface of the basin to promote better infiltration. As a result, the riprap should not be used to form a pilot channel. Instead, the riprap should terminate in a broad apron that serves as a crude level spreader (see Figure 6.1).

Basin Slopes

It is very important to grade the floor of the basin to have a slope close to zero. Unlike detention ponds, the objective in infiltration basin design is to achieve a uniform ponding depth across the entire surface of the basin. If the basin is sloped toward the riser, or if low spots are created, storm runoff will concentrate only in a small portion of the basin. Since the soil has only a limited infiltration capacity, these low spots will remain under water for a longer time, and may become chronically wet. Over a period of time, the enhanced deposition of sediment in low areas may clog the surface soils.

The side-slopes of the basin should be no steeper than 3:1 (h:v) to allow for proper vegetative stabilization, as it is extremely difficult to establish and maintain erosion resistant ground covers on steep slopes. Gentle slopes also allow for easier mowing and access, and better public safety.

Establishing Vegetation

A dense turf of water tolerant grass should be established on the floor and side-slopes of infiltration basins immediately after construction. The turf promotes better pollutant removal in several ways. First, root penetration and thatch formation by the turf maintains and sometimes even improves the original infiltration capacity of the basin floor. Secondly, the turf grows through the accumulated pollutants that are deposited within the basin, preventing their resuspension during larger storms. Thirdly, the turf takes up soluble nutrients for growth, and converts then into less available particulate forms. If clippings are bagged or raked as part of routine mowing operations, the plant nutrients can effectively be removed from the system. Finally, a dense growth of turf will prevent soil erosion and basin scouring that could negate the removal efficiency of the basin. Ground covers such as tall fescues and bermuda grass are generally used for this purpose.

Trees and shrub plantings can achieve many of the same goals as ground covers, but may pose a maintenance problem if the basin floor will frequently be mowed or tilled.

Nature of Soils

As noted earlier, the greatest amount of nutrient and metal sorption occurs in soils that typically have the least capacity to infiltrate runoff. One notable exception are soils with a high content of organic matter, which provide an abundance of binding sites for pollutants. Basin sites with poor organic soils can be improved over time by the natural thatch formation of the turf. The organic matter content can be improved by tilling plant residues below ground during normal maintenance tilling operations.

Maximum Draining Time

The depth of exfiltration storage within the basin needs to be adjusted so that it completely drains within three days after the maximum design exfiltration event. The appropriate design techniques are discussed in Md WRA (1984). Complete drainage is needed to maintain aerobic conditions in the soil profile within the basin long enough to favor bacteria that aid in pollutant removal (and also to ensure that the basin will be empty in time for the next storm).

If a basin is constructed over soils with marginal infiltration capacity (silt loams or loams), it is prudent to adjust the depth of exfiltration storage to completely drain within two days. Recent experience has shown that optimistic projections of future infiltration capacity on marginal sites can lead to chronic standing water problems.

Minimum Draining Time

Moderate to poor pollutant removal has been observed in partial exfiltration systems that detain water for less than 6 hours (MWCOG, 1983b). Short residence times do not allow for adequate exfiltration which, in turn, limits pollutant removal capability. Several design factors can lead to too rapid a passage of runoff through a basin. One of the most frequent problems involves setting the elevation and diameter of the low flow orifice. If the orifice diameter is too wide, small runoff events pass through the basin too quickly to achieve any storage/exfiltration. As a consequence, the pollutant removal capability of the basin will be diminished. Conversely, if the low flow orifice diameter is too narrow, the designer runs the risk of creating a quasi-permanent pool. The backed up water diminishes the exfiltration capacity of the basin, and is likely to produce a host of nuisance and maintenance problems as well.

This design problem becomes even more difficult for basins serving large areas which have sustained baseflows that must be passed through the basin. In such cases, the low-flow orifice must be set so that runoff is retained but baseflow is not allowed to back up in the basin, a problem which would significantly reduce available storage and exfiltration capacity between storms. This particular problem can be circumvented by using either the side-by-side or off-line design variations. In most cases, the existence of baseflow at a potential basin location is a good tip-off that the site may not be suitable for infiltration. A wet pond or extended detention pond may prove to be a better alternative on these sites.

Sediment Forebays

The longevity of an infiltration basin can be enhanced if sediment forebays are constructed near the inlets to trap incoming sediment loads. Forebays are also an important design element because they help to reduce incoming water velocity, and distribute it more evenly across the basin floor.

PHYSICAL SUITABILITY AT THE SITE LEVEL

Development sites should be carefully evaluated to be certain that they are actually capable of disposing runoff via exfiltration. The following factors should be examined early in the site-planning stage to adequately screen the feasibility of the site. More detailed guidance on feasibility tests for basins can be found in Md WRA (1984).

Soils

Basin are not a feasible option on sites with "D" soils (infiltration rates of less than 0.27 inches per hour), or any soil with a clay content greater than 30% (as determined from the SCS soil textural triangle). Silt loams and sandy clay loams ("C" soils) provide marginal infiltration rates, and should probably not be considered for basin applications in most circumstances (Table 6.2). Soils with a combined silt/clay percentage of over 40% by weight are susceptible to frost-heave, and are not good candidates for infiltration basin applications. Also, basin are unsuitable if the site is located over fill soils that form an unstable subgrade, and are prone to slope failure.

If the soils at a site pass these preliminary tests, an additional series of soil cores or trenches should be gathered to a depth at least five feet below the elevation of the basin floor. Because soil conditions vary substantially over short distances, up to 6 cores per trench may be needed at each site to adequately characterize future infiltration capacity. These should be examined for evidence of any impermeable soil strata that might impede infiltration, such as localized clay lenses, hardpans, or fragipans. The presence of such layers do not necessarily preclude a basin, as long as it penetrates them completely.

Slope

Infiltration basins are not feasible if the slope of the contributing watershed is greater than 20%. Within the basin itself, a slope of less than 5% is preferable.

Depth to Bedrock

At least four feet of clearance will be needed between the floor of the basin and the bedrock level. This data can be obtained from local soil maps and should always be confirmed with soil test borings.

Depth to Seasonally High Water Table

A minimum of two to four feet of clearance is needed between the floor of the basin and the seasonally high water table. This depth can be readily determined from soil borings taken during wet weather. High water tables often present a major obstacle to the use of infiltration basins, since basins are usually located in depressions at the low end of a watershed where local water tables are located near the the ground surface.

Proximity to Wells and Foundations

Basins should be located at least 100 feet away from drinking water wells to minimize the possibility of groundwater contamination, and should be situated at least 10 feet down-gradient and 100 feet up-gradient from building foundations to avoid potential seepage problems.

Maximum Depth of Reservoir

To insure that the basin completely drains within 72 hours, it may be necessary to limit the depth of the basin if underlying soils have relatively low exfiltration rates. Recommended depth limits for basins are shown for various soil textures in Table 6.2.

Watershed Size

Md WRA (1983b) suggests that basins can be applied to sites ranging from 5 to 50 acres in size. Other BMPs, such as extended detention ponds and wet ponds, are better candidates on larger sites as they are more capable of handling sustained baseflow.

Table 6.2: Soil Limitations For Infiltration Basins

SOIL TYPE	MINIMUM INFIL-TRATION RATE (fc--inches/hr)	SCS SOIL[1] GROUP	MAXIMUM DEPTH OF[2] STORAGE (inches) 48 hrs	72 hrs
Sand	8.27	A	397	595
Loamy Sand	2.41	A	116	174
Sandy Loam	1.02	B	49	73
Loam	0.52	B	25	37
Silt Loam	0.27	C	13	19

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[1] Sandy Clay Loams, Clay Loams, Silty Clay Loams, Sandy Clay, Silty Clay, and Clay Soils are not included as these soil types are all NOT FEASIBLE for infiltration basins.

[2] Maximum Depth in the Basin that can drain completely within 48 or 72 hours after a storm, given the soil infiltration rate.

INFILTRATION BASIN COSTS

Predicting Infiltration Basin Costs

Because so few infiltration basins have been built, there is not enough data to develop specific cost projections for this practice. However, given the many similarities in design and construction methods between infiltration basins and dry ponds, it is reasonable to assume that the dry pond cost equation (Wiegand et al., 1986) can be used as a surrogate measure of cost. Thus, for infiltration basins greater than 10,000 cubic feet in volume, the following equation can be used to predict cost, until better infiltration basin cost data becomes available:

(EQ 6.1) C = 10.7 Vs**0.69

where: C = construction cost in 1985 dollars.
Vs = storage volume up to the crest of the emergency spillway in the basin (including any dead storage reserved for exfiltration purposes).

The cost equation does not include additional costs for land acquisition (if any) or for any sediment trapping structures. An additional 25% should be added to the cost estimate to account for contingencies involved in planning, design, and administration.

One cost advantage unique to infiltration basins (in comparison with other infiltration practices) is that they can serve as temporary sediment basins during the construction phase of development, thereby fulfilling both stormwater and erosion control requirements at one time. However, if a basin is used as a temporary sediment control facility, it should not be excavated to more than two feet above the final elevation of the basin floor, so that the infiltration capacity is preserved.

Cost-Effectiveness

Infiltration/detention basins seldom cost much more than conventional dry ponds. Some extra costs are incurred to provide extra dead storage needed for exfiltration, but this storage is usually small in relation to that needed for stormwater management purposes. As a result of these economies, infiltration/detention basins are one of the most cost-effective water quality BMPs available.

The full infiltration basin design will cost somewhat more since a larger storage volume must be reserved to store the entire runoff hydrograph associated with the design storm. Cost projections prepared over a wide range of land uses and watershed areas (Wiegand et al., 1986), however, indicate that full infiltration basins may only cost 10 to 20% more than conventional dry ponds, which still makes them a competitive BMP option.

Infiltration basins also exhibit economies of scale with regard to construction costs. This means that it costs less per unit volume to build an infiltration basin on a large watershed than on a small one, or on commercial rather than residential developments.

CONSTRUCTION SPECIFICATIONS AND MAINTENANCE REQUIREMENTS

Proper construction and routine maintenance are extremely important for sucessful infiltration basin implementation. Initial field reports suggest that basins appear to fail at a higher rate than other infiltration practices. In a recent survey conducted by Md WRA (1986b), approximately 40% of the infiltration basins sampled had partially or totally clogged within the first few years of operation. Moreover, many of the structures failed almost immediately after completion or never worked properly from the outset. The most common problem has been the partial or total loss of infiltration capacity, typified by the presence of standing water for long periods of time. In most instances, basin failure was primarily due to inadequate field testing of soil infiltration rates, prior use as a sediment basin, compaction by heavy equipment, or poor upland sediment control practices. The discussion below highlights construction and maintenance procedures that can prevent or at least alleviate premature surface clogging.

Construction Specifications

1. Before the development site is graded, the area planned for the basin should be roped off to prevent heavy equipment from compacting the underlying soils.

2. If the basin is not designated for sediment control, diversion berms should be placed around its perimeter during all phases of construction. Sediment and erosion control plans for the site should be oriented to keep sediment and runoff completely away from the basin. Actual construction of the basin should not begin until after the site has been completely stabilized.

3. If the basin is to be used as a temporary sediment basin during the construction phase, it should only be excavated to within two feet of the final design elevation of the basin floor. Sediment which accumulates during the construction phase can then be removed when the basin undergoes final excavation after the development has been completed.

4. The basin should be excavated using light earth-moving equipment with tracks or over-sized tires. Normal rubber tires should be avoided since they compact the subsoil and reduce its infiltration capabilities. For the same reason, the use of bulldozers or front end-loaders should be avoided. Since some compaction of the underlying soils is still likely to occur during excavation, the floor of the basin should be deeply tilled with a rotary tiller or disc harrow. Several passes with a leveling drag should can then be made to smooth out the basin floor.

5. The basin embankment and inlet/outlet channels should be constructed following local pond specifications, such as core trenches and anti-seep collars (Md SCS, 1976).

6. The basin should be stabilized with vegetation within a week after construction. Use of low maintenance, rapid germinating grasses such as fescues are recommended. The condition of the newly established vegetation should be checked several times over the first two months, and any necessary remedial actions taken (e.g., reseeding, fertilization,and irrigation).

Routine Maintenance

The maintenance required for infiltration basins is slightly greater than that needed for dry detention ponds. Some of the normal maintenance tasks for infiltration basins are detailed below.

INSPECTION

The performance of the infiltration basin should be checked after every major storm in the first few months after construction. Particular attention should be paid to how long runoff remains in the structure. Standing water in the basin within 48 to 72 hours after a storm, is a good indication that the infiltration capacity of the basin may have been overestimated. The inspector should look for factors which may be responsible for clogging the basin, such as upland sediment erosion, low spots, excessive compaction, or marginal soils; and then get the contractor to make any needed repairs. As a practical matter, local governments should not release any bonds posted for a basin until the inspector determines that it is performing as designed.

Thereafter, the basin should be inspected annually. Some of the more important items to check for include: differential settlement, cracking, erosion, leakage or tree growth on the embankment; the condition of the riprap in the inlet, outlet and pilot channels; sediment accumulation in the basin; and the vigor and density of the grass turf on the floor of the basin.

MOWING

The buffer, side-slopes, and basin floor must be mowed at least twice a year to prevent woody growth. More frequent mowing may be needed if the basin is to be used as a passive recreation area. Mowing operations may be difficult since the basin floor may often be soggy. If a low maintenance grass such as Tall Fescue is used, basin mowing can be performed in the normally dry months of June and September (Md WRA, 1983a).

DEBRIS AND LITTER REMOVAL

Trash will tend to collect in full-infiltration basins since they do not have outlets. Infiltration/detention designs also will collect trash that might clog the riser or low flow orifice. Therefore, it is a good practice to remove all debris and litter during each mowing operation.

EROSION CONTROL

This is a very important maintenance task since eroded sediments can adversely affect the infiltration capacity of a basin. Eroding or barren areas should be immediately revegetated.

TILLING

If a basin is located on marginally permeable soils, annual or semi-annual tilling operations may be needed to maintain infiltration capacity. A rotary tiller or disc harrow can be used, preferably in the late summer months when soil permeability is likely to be the lowest (freezing and thawing of the soil in the winter and spring months often helps to break up the soil and keep infiltration capacity high). Tilled areas should be immediately revegetated to prevent erosion.

Non-Routine Maintenance

STRUCTURAL REPAIRS/REPLACEMENT

If the basin is of the infiltration/detention basin design, the pipes and barrels will eventually need to be replaced. Corrugated metal pipe has an estimated longevity of approximately 25 years in the field; whereas, concrete pipe and other structures may last up to 50 years. However, if the basin is designed for full exfiltration (i.e., no outlet apart from an earthen emergency spillway) then the frequency and cost of structural repairs is sharply reduced.

RESTORATION OF INFILTRATION CAPACITY.

Over time, the original infiltration capacity of the basin floor will gradually be lost. If the problem has been caused by surface clogging (e.g., sediment accumulation or local compaction), deep tilling can be used to break up the clogged surface layer, followed by regrading and leveling. Md WRA (1986b) suggests that deep tilling may be needed every 5 to 10 years. In some instances, sand or organic matter can be tilled into the basin soils to restore infiltration capacity as well. If a basin still experiences chronic problems with standing water after these measures have been taken, it is likely that the original infiltration capacity was overestimated. It may then be necessary to install perforated underdrains beneath the basin to remove the excess water.

SEDIMENT REMOVAL.

Infiltration basins are normally located in smaller residential watersheds that do not generate large sediment loads, or are equipped with some kind of sediment trap. However, even though sediment loads to the basin are likely to be low, they will still have a negative impact on basin performance, since the sediment deposits will reduce the storage capacity reserved for exfiltration and may also clog the surface soils.

Sediment removal methods in infiltration basins are different from those utilized for extended detention and wet ponds. Removal should not begin until the basin has had a chance to thoroughly dry out, preferably to the point where the top layer begins to crack. The top layer should then be removed by light equipment, taking care not to unduly compact the basin floor. The remaining soil can then be deeply tilled with a rotary tiller or disc harrow to restore infiltration rates. Areas disturbed during sediment removal should be revegetated immediately to prevent erosion.

Total Maintenance Costs

Infiltration basins have only recently come into widespread use in the Washington region, and consequently, there is very little data on which to base maintenance cost projections. However, since the routine and non-routine maintenance tasks for infiltration basins appear to be similar to those associated with conventional dry detention ponds, it may be reasonable to assume that annual maintenance costs (routine and non-routine) will comprise 3-5% of a basin's initial construction cost (Wiegand et al., 1986). This should be considered an interim estimate until more infiltration basin maintenance experience has been gained.

ENVIRONMENTAL ATTRIBUTES OF BASINS

Impacts to the Natural Environment

Infiltration basins are probably the best available BMP for protecting downstream aquatic life. Basins help to maintain the pre-development water balance at the site, minimize streambank erosion, filter out pollutants and augment low flows during the summer months. In addition, infiltration basins do not produce thermal or low dissolved oxygen impacts that can be associated with wet ponds.

The value of infiltration basins in creating local wildlife habitat, however, is not as great as for wet or extended detention ponds. This is largely because the floor in most basins is managed to maintain a dense growth of turf and, as a result, the food and cover supplied to wildlife is poor. However, the perimeter of the basin may be planted with trees and shrubs that provide better wildlife habitat. Suggested species can be found in the basin landscaping guide provided in Chapter 9.

One potential negative impact of basins is the risk of groundwater contamination. Long-term studies of pollutant migration in soils underneath various infiltration practices indicate only limited downward migration of pollutants through the soil (US EPA, 1983; OWML, 1982). Possible exceptions include very soluble pollutants such as nitrate, chlorides and gasoline. Nightingale (1987) found no evidence of groundwater contamination underneath five infiltration basins in California that had been in operation for 5-20 years. A more definitive assessment of the possible risks of groundwater contamination by infiltration is the focus of a current monitoring survey being conducted in Maryland.

Impacts on the Human Environment

While infiltration basins do not provide all the amenities associated with wet ponds, they can look attractive when they are well landscaped, naturally contoured and frequently maintained. In some cases, the basin can be utilized for recreation (e.g., ball fields and playgrounds).

The importance of regular maintenance cannot be overstated. Lack of regular maintenance can quickly turn an infiltration basin into an community eyesore. Standing water may breed mosquitos or create undesirable odors (Hantzsche and Franzini, 1980), and may kill the turf on the basin floor.

RELEVANT DESIGN GUIDANCE

A summary of design criteria for infiltration basins is provided in Table 6.3. In addition, the following references should be consulted for more detailed guidance on the design and construction of infiltration basins.

Maryland Water Resources Administration, 1984. Standards and Specifications for Infiltration Practices. Annapolis, MD.

Maryland Water Resources Administration, 1985b. Inspectors Guideline Manual for Stormwater Management Infiltration Practices. Annapolis, MD.

DESIGN SUMMARY: INFILTRATION BASINS

- **SITE EVALUATION:**
  Soils must be tested prior to design to ensure that the site is capable of infiltration. Since soil characteristics vary spatially, a minimum of three soil borings and/or trenchings should made within the basin. Each core should extend at least five feet below the anticipated floor of the basin. Soils within this zone (0-5 feet below basin floor) should have a minimum field infiltration rate of 0.5 inches/hr (fc), and be above the seasonally high water table and bedrock level. Basins should never be constructed over fill soils.

- **WATERSHED SIZE:**
  Full exfiltration basins can be applied on small watersheds (5 to 25 acres) that do not have a permanent source of baseflow. Infiltration/detention basins can be used on larger watersheds (up to 50 acres) if there is a design feature for routing baseflow through the structure without infiltrating (side-by-side design).

- **DEGREE OF EXFILTRATION:**
  To achieve significant pollutant removal and downstream channel protection, the basin should be capable of completely exfiltrating the first half inch of runoff per contributing impervious acre. When possible, even greater quantities of exfiltration are preferable.

- **SHAPE OF BASIN:**
  The floor of the basin should be graded as flat as possible to permit uniform ponding and exfiltration. Low spots and depressions should be leveled out. Side-slopes leading to the floor should have a maximum slope of 3:1 (h:v) to allow for easier mowing and better bank stabilization.

- **CONSTRUCTION:**
  The basin should be excavated with light equipment equipped with tracks or over-sized tires to minimize compaction of the underlying soils. After the basin is excavated to the final design elevation, the floor should be deeply tilled with a rotary tiller or disc harrow to restore infiltration rates, followed by a pass with a leveling drag. Vegetation should be established immediately. The riser, embankment, and emergency spillway should be sized and constructed to the normal specifications for conventional ponds.

- **VEGETATION:**
  The floor of the basin should be stabilized by a dense turf of water tolerant reed canary grass or tall fescue, immediately after basin construction. The grass turf promotes better infiltration, pollutant filtering, and prevents erosion of the basin floor.

- **BASIN INLETS:**
  All basins should have sediment forebays or riprap aprons that dissipate the velocity of incoming runoff, spread out the flow and trap sediments before they reach the basin floor.

- **INLET/OUTLET INVERT ELEVATIONS:**
  The storm drain inlet pipe (or channel) leading to the basin should discharge at the same invert elevation as the basin floor. Similarly, the low flow orifice in infiltration/detention basins should be set at the same elevation as the basin floor, to prevent baseflow from ponding and thus impeding the function of the basin.

- **MAXIMUM DRAINING TIME:**
  As a general rule, the depth of storage should be adjusted so that the basin completely drains within 72 hours. On sites with marginal soils, or basins with a large floor, it is prudent to design the basin to drain within 48 hours. This can be accomplished by increasing the surface area of the basin floor, or by reducing the depth of storage, or both. infiltration/detention basins is sized too large.

- **BASIN BUFFER:**
  A minimum buffer of 25 feet from the edge of the basin floor to the nearest adjacent lot should be reserved. A landscaping plan should be prepared for the basin buffer that emphasizes the use of low maintenance, water tolerant, native plant species that provide food and cover for wildlife, and when necessary, can act as a screen.

- **INSPECTIONS:**
  The change in standing water depth above the basin floor over time should be checked after each major storm in the first few months after basin construction to monitor exfiltration rates. Similar tests should be conducted annually to gage the degree of surface clogging that may occur over the years, and to help in scheduling restorative deep tilling operations.

- **EROSION CONTROL:**
  Infiltration basins can be used as temporary sediment control basins during the construction phase, as long as at least two feet of original soil is preserved (that will be excavated later for the basin). As with all infiltration facilities, upland construction areas should be completely stabilized prior to permanent basin construction.

- **ACCESS:**
  Adequate access to the basin floor should be provided from public or private right-of-way that can withstand light equipment. Such access should be at least 12 feet wide, and should not cross the emergency spillway.

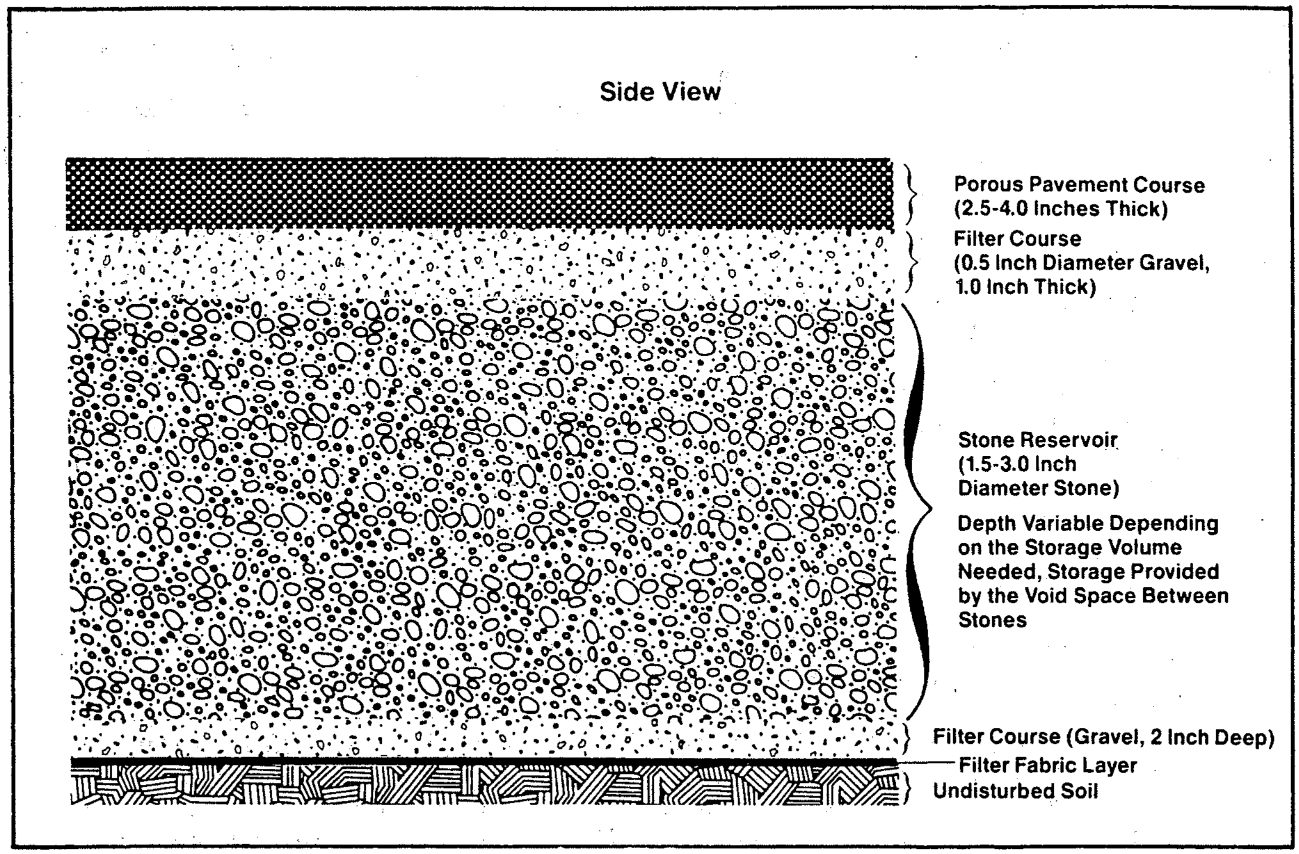
- **MAINTENANCE:**
  Maintenance responsibilities should be clearly vested, and funds reserved for both routine and non-routine maintenance tasks. Wet-weather inspections, with as-built plans in hand, should be conducted annually. The basin floor is best maintained as wet meadow, and should be mowed twice a year to prevent woody growth. If standing water becomes a problem over time due to gradual surface clogging, infiltration rates can be restored by deep tilling operations. If tilling does not solve the problem, it may be necessary to convert the basin into a wet pond or shallow marsh, or install underdrains to collect the water.

CHAPTER 7: POROUS PAVEMENT

Porous pavement has a high capability to remove both soluble and fine particulate pollutants in urban runoff, and also provides groundwater recharge, low flow augmentation, and streambank erosion control. Its use is generally restricted to low volume parking areas, although it can accept runoff from rooftop storage or adjacent conventionally paved areas. As a BMP, porous pavement is only feasible on sites with gentle slopes, permeable soils, and relatively deep water table and bedrock levels. When these conditions are met, porous pavement is a reasonably cost-effective BMP, particularly if off-site runoff contributions are not great.

When properly designed and carefully installed, porous pavement has load bearing strength, longevity, and maintenance requirements similar to conventional pavement. Some other advantages of porous pavement are reduced land consumption, reduction or elimination of the need for curb and gutters and downstream conveyance systems, the preservation of the natural water balance at the site, and a safer driving surface which offers better skid resistance and reduced hydroplaning.

Figure 7.1: Schematic of Typical Porous Pavement Section



Source: City of Rockville (1984a)

The major drawback associated with porous pavement is that if it becomes clogged it is difficult and costly to rehabilitate. The risk of premature clogging of the pavement is fairly high, and can be prevented only if sediment is kept off of the pavement before, during and after construction. Other disadvantages include the need for extensive feasibility tests, inspections, very high levels of construction workmanship (which cannot always be assured), and a possible risk of groundwater contamination (probably slight).

METHODS USED FOR POROUS PAVEMENT

A typical cross-section of porous pavement is shown in Figure 7.1. Runoff rapidly infiltrates through the pores of the 2-4 inch porous asphalt layer into the void spaces of an underground stone reservoir. The reservoir is composed of two layers: a one-inch filter course of half-inch diameter gravel placed over a deeper reservoir course of 1.5-3.0 inch diameter stone. Runoff then exfiltrates out of the stone reservoir and into the underlying subsoil or is collected by perforated underdrain pipes and routed to an outflow facility. Thus, the storage capacity of porous pavement is primarily a function of the depth of the underground reservoir (plus any runoff lost via exfiltration through the subsoils).

Under normal conditions, the porous asphalt layer merely acts as a rapid conduit for runoff to reach the stone reservoir (typical infiltration rates for open-graded porous asphalt are in excess of 150 inches/hour). A less preferable alternative for directing runoff into the stone reservoir is to install drop inlets or drill holes through a layer of conventional asphalt.

Porous pavement designs fall into three basic categories, based on the runoff storage provided by the stone reservoir and the degree of reliance on exfiltration. These are described below and are illustrated in Figure 7.2.

Full Exfiltration System

With this design, the only way runoff can exit the stone reservoir is to exfiltrate through the underlying subsoil (i.e., there is no positive pipe oulet draining the stone reservoir). Consequently, the stone reservoir must be large enough to accommodate the entire increase in runoff volume for the design storm , less any runoff volume which is exfiltrated during a storm. The complete exfiltration system provides total peak discharge, volume, and water quality control for all rainfall events less than or equal to the design storm. An emergency overflow channel (such as a raised curb) is located above-ground to handle the excess runoff from storms greater than the design storm.

Partial Exfiltration System (stone filtration system)

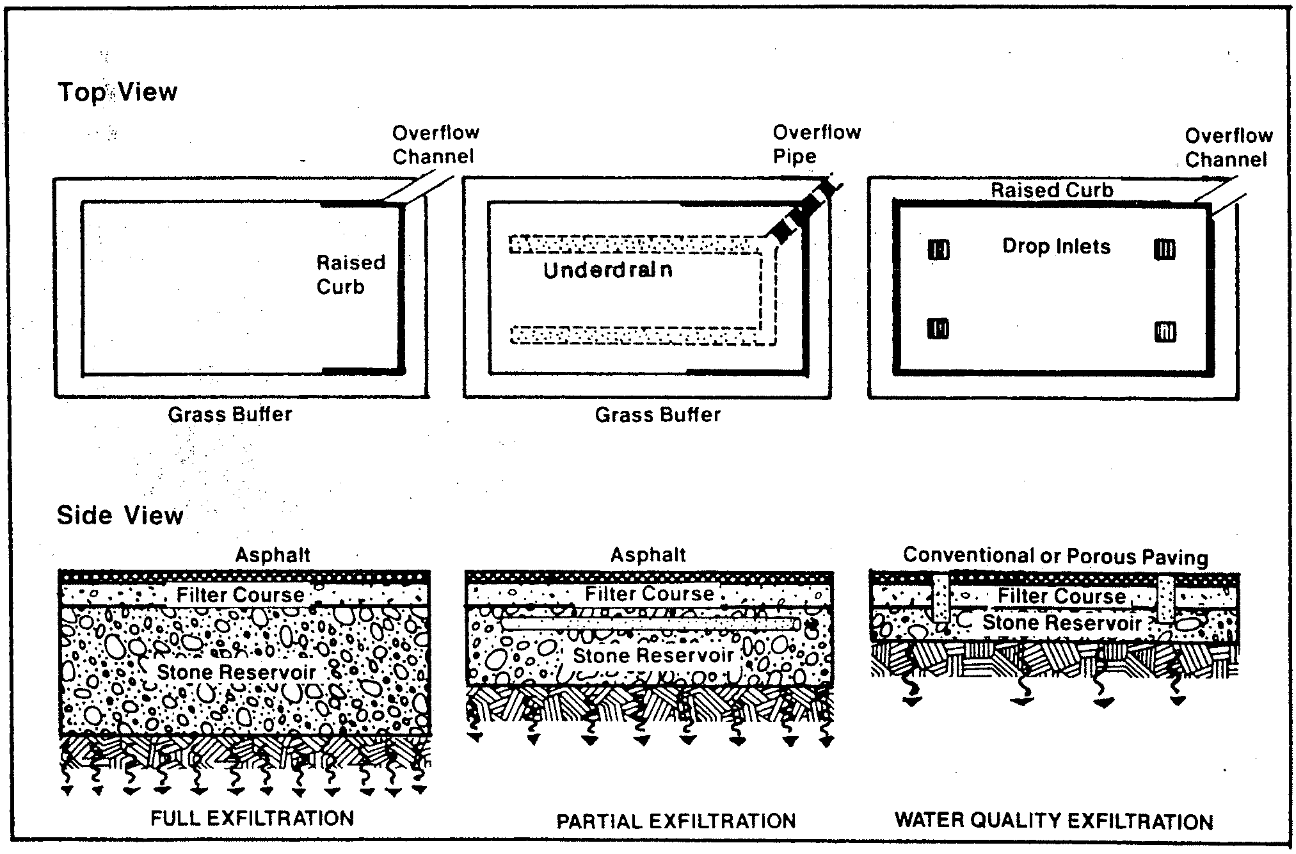
It may not always be feasible or prudent to totally rely on exfiltration to dispose of runoff. For example, there may be concerns about the long term permeability of the underlying soils, downstream seepage, or clogging at the interface between the filter fabric and subsoil. In these situations, an underground drainage system can be installed, comprised of regularly spaced perforated pipes located in shallow depressions that collect the runoff and direct it to a central outlet. The size and spacing of the underdrain network is set to pass the two year storm. However, most of the runoff volume from smaller storms will still be exfiltrated before it is collected, thereby providing significant water quality control.

An alternative method of controlling the design storm in partial exfiltration systems is to place perforated pipes (on the underside only) near the top of the stone reservoir (NVPDC, 1987). Runoff then must entirely fill up the stone reservoir before it is discharged from the facility. This design should promote a greater degree of exfiltration, particularly for smaller storms.

Water Quality Exfiltration System

With the water quality design, the storage volume of the stone reservoir is set to only handle the first flush of runoff volume during a storm. The first flush has been variously defined as 1) one-half inch of runoff per contributing impervious acre, 2) one-half inch runoff per contributing total acres and 3) the volume of runoff produced by a one-inch storm. Runoff volumes in excess of the first flush are not treated by the system, and instead, are conveyed to a conventional stormwater management facility further downstream. Water quality exfiltration system will not satisfy stormwater storage requirements, but may result in smaller, less costly facilities downstream. In most sites, the first flush runoff volume can fit within the normal six-inch layer of stone aggregate required for conventional paving. Slot or drop inlets through conventional asphalt can be used in addition to porous asphalt to route the first flush into the stone reservoir.

Figure 7.2: Comparison of Selected Exfiltration Systems For Porous Pavement



A design variation that enables a porous pavement site to accept runoff contributed from off-site areas is shown in Figure 7.3. As shown, a series of underground perforated inflow pipes are used to convey runoff into the porous pavement and evenly distribute it throughout the stone reservoir. In addition, a pretreatment facility is needed to remove sediment, oil and grit before it reaches the reservoir. Some useful pretreatment techniques for porous pavement are shown in Figure 7.4.

Figure 7.3: Design Technique For Accepting Off-Site Runoff

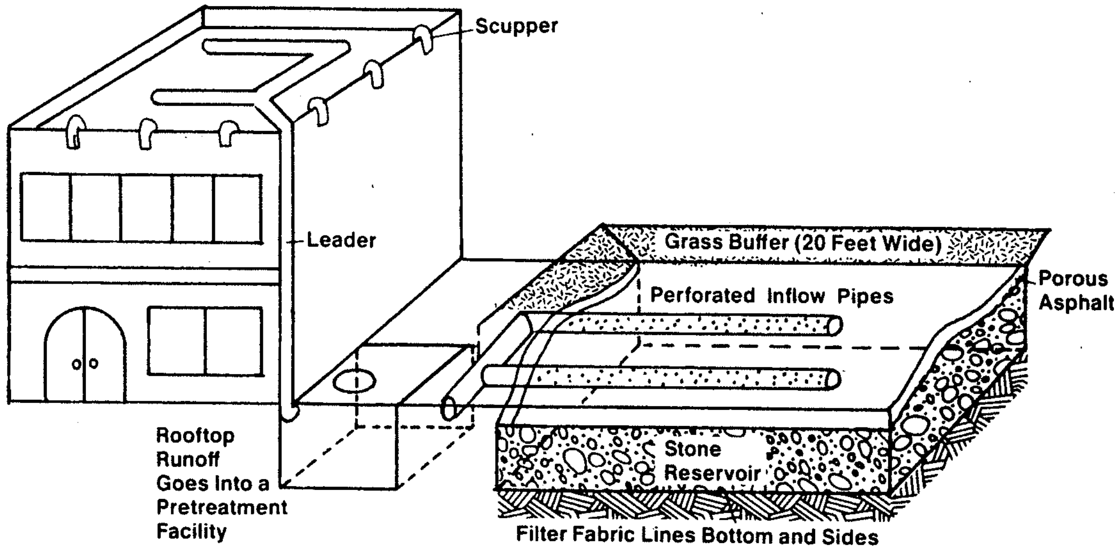
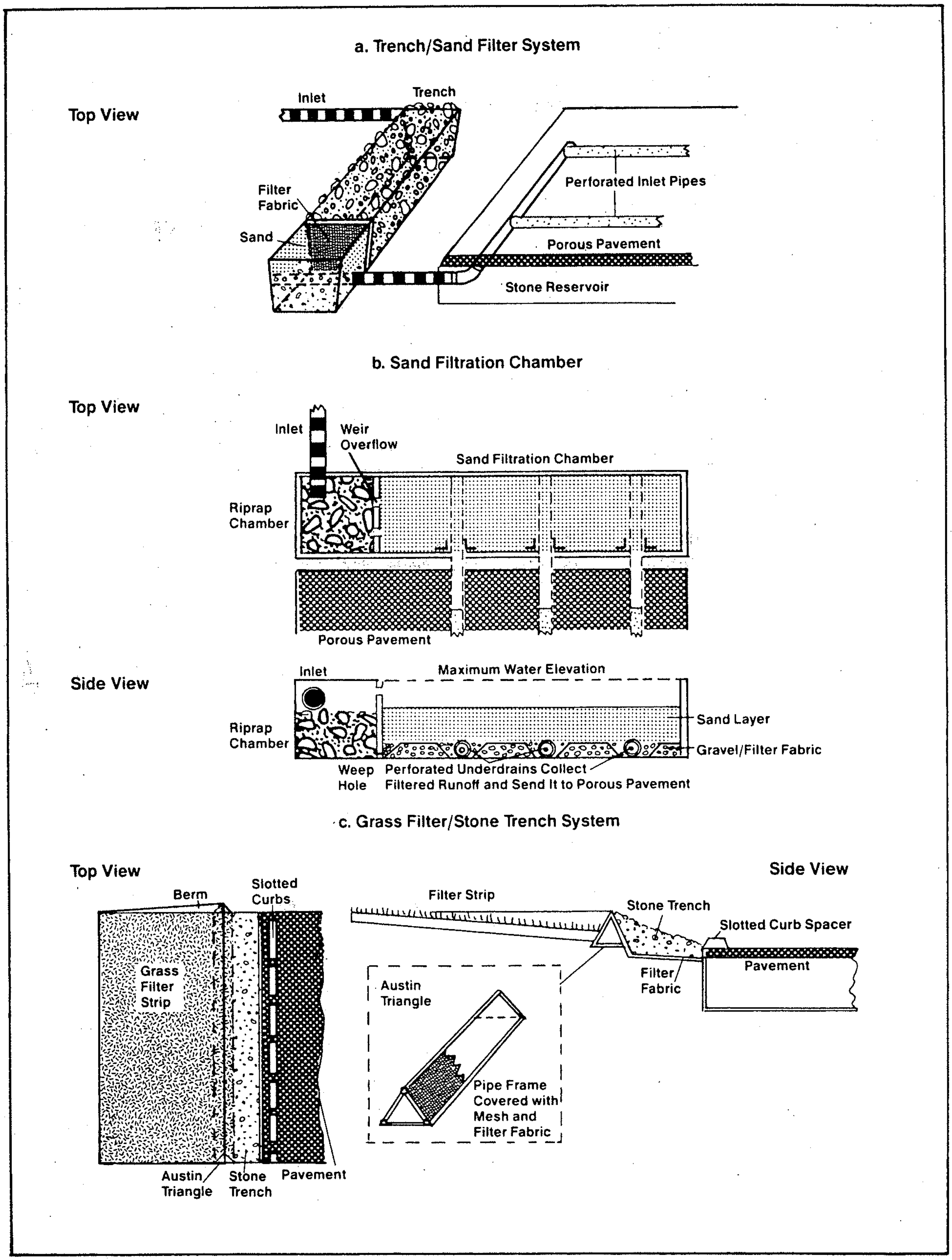


Figure 7.4: Pretreatment Methods For Porous Pavement Sites



EFFECTIVENESS IN STORMWATER CONTROL

Porous pavement is unique in that it can almost completely reproduce the natural, pre-development hydrologic regimen at a site.

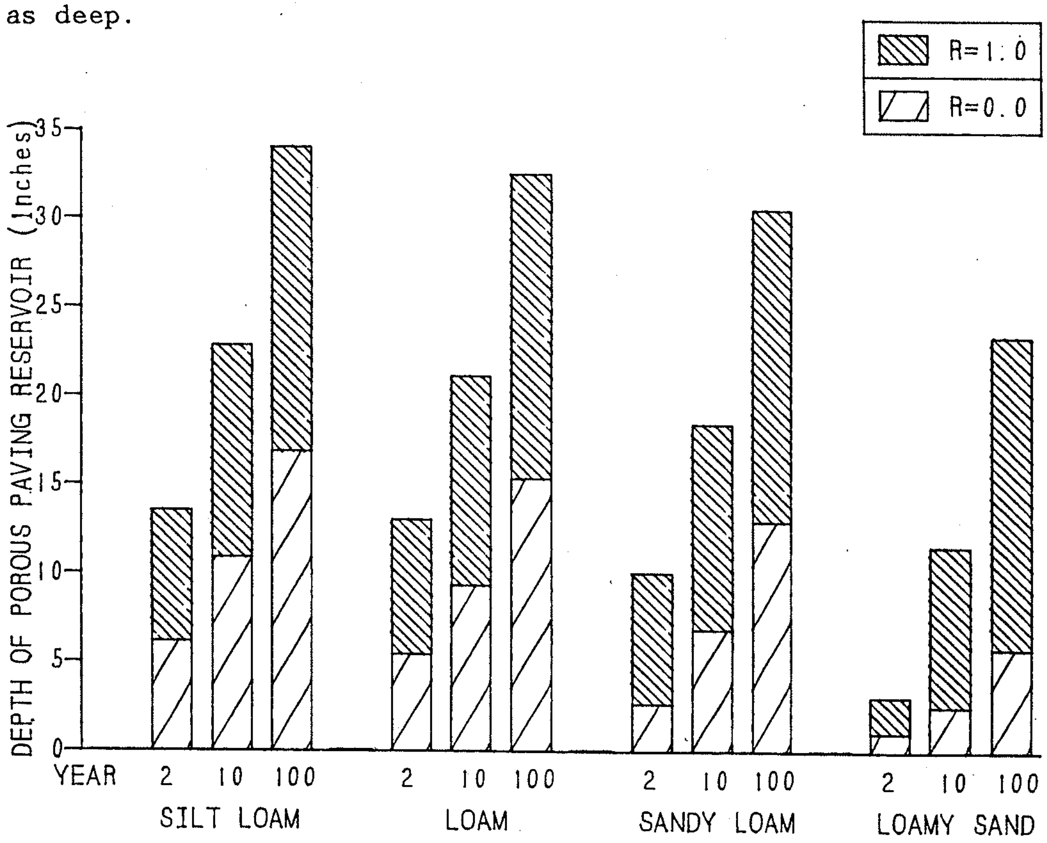
Peak Discharge Control

Both complete and partial exfiltration systems can control peak discharges to pre-development levels for both the design storm and smaller storms. The only limitation of either porous pavement system is that there may not be sufficient depth for the stone reservoir to accommodate the runoff volume from large (10 to 100 year) design storms, particularly if there are large off-site areas contributing runoff to the pavement. The required depth for the stone reservoir can be calculated using Md WRA (1984) design equations.

Figure 7.5 presents graphical solutions for stone reservoir depth based on capturing the entire runoff volume from contributing impervious areas for selected design storms and soil conditions, assuming full exfiltration. As can be seen, stone reservoirs up to 3 feet deep may be needed in some circumstances. Apart from requiring a hefty investment in stone, these deep reservoirs may be too close to the water table or bedrock to be feasible. In such cases, partial exfiltration systems may be a desirable alternative, since they normally require shallower stone reservoirs.

Figure 7.5: **Minimum Stone Reservoir Depths For Full Exfiltration Porous Pavement Systems, For Selected Design Storms and Soils**

NOTE: Partial exfiltration and water quality designs will not be as deep.



Groundwater Recharge

Water balance studies of porous pavement sites in Willow Grove, Pennsylvania (Gburek and Urban, 1980), Rockville, Maryland (MWCOG, 1983b), and Prince William County, Virginia (OWML, 1986b) indicate that 60-90% of the annual rainfall volume is diverted to groundwater. Groundwater recharge rates are slightly higher under porous pavement than under natural conditions (Gburek and Urban, 1980), as vegetation is absent and soil water is not transpired during the summer months.

Enhanced groundwater recharge is very important since it maintains flow levels in small headwater streams during critical dry weather periods. Even moderate levels of development have been shown to drastically reduce summer baseflow levels in small and mid-sized streams, with adverse consequences to water quality and aquatic habitat. Md WRA (1986b) estimates that even the smaller sized water quality exfiltration systems are capable of maintaining summer baseflow levels to within 90% of natural, pre-development levels.

Volume Control

Unlike detention or retention ponds, porous pavement reduces the volume of storm runoff produced following development. Only about 20-40% of storm runoff that entered the Willow Grove, Rockville, and Prince William porous pavement sites emerged again as surface runoff. The remainder either evaporated within the structures or was diverted to groundwater recharge.

Streambank Erosion Control

The superior hydrologic performance of complete and partial exfiltration systems should prevent serious streambank erosion immediately below the site. However, since porous pavement normally only serves a small portion of a stream's watershed, other practices (such as extended detention or infiltration basins) may need to be installed elsewhere in the basin to provide a comprehensive level of streambank protection.

POLLUTANT REMOVAL

Field studies have demonstrated that partial exfiltration systems are capable of achieving high levels of removal of both soluble and particulate pollutants. It should be emphasized, however, that porous pavement IS NOT intended to remove coarse particulate pollutants, as they can rapidly clog asphalt and filter cloth pores. Because of the cost and difficulty of rehabilitating clogged pavement, every effort should be made to keep coarse grained particles from ever entering the surface of the pavement.

Porous pavement is primarily designed to remove pollutants deposited on the pavement surface from the atmosphere. These pollutants are normally either very fine grained or are soluble, and should not normally present any problems with respect to clogging. The annual rate of atmospheric pollutant deposition is considerable (Table 7.1), and accounts for most, if not all, of the pollutant export from completely impervious surfaces (MWCOG, 1983b).

Table 7.1: Average Annual Atmospheric Deposition Rates for the Washington, D.C. Area

POLLUTANT	RURAL (a)	SUBURBAN (b) (lbs/acre/year)	URBAN (c)
Total Solids	99	155	245
Chemical Oxygen Demand	199	133	210
Total Nitrogen	19.9	12.8	17.0
Nitrate-N	9.4	5.6	6.8
Ammonia-N	5.5	1.1	1.0
Total Kjeldahl N	10.5	7.2	10.2
Total Phosphorus	0.71	0.50	0.84
Ortho-phosphorus	0.28	0.26	0.35
Trace Metals			
Cadmium	ND	0.09	0.003
Copper	ND	0.21	0.61
Lead	0.06	0.44	0.53
Iron	ND	1.57	5.60
Zinc	0.67	1.35	0.65

Source: MWCOG (1983b). Note: ND = no data

Pollutant Removal Mechanisms of Porous Pavement

Most of the pollutant removal in a porous pavement site is accomplished after the runoff has exfiltrated through the stone reservoir and into the underlying soil (except for the undesirable trapping of particulates in the asphalt pores or stone reservoir). Thus, the degree of pollutant removal achieved in porous pavement is closely related to the amount of runoff that is actually exfiltrated into the soil.

SORPTION

Some soluble forms of pollutants such as ortho-phosphorus and zinc become attached to binding sites on soil particles as they pass through the soil layer. Most of the sorption occurs within the first foot of soil, and is bound up for long periods of time (US EPA, 1977). The greatest sorption of nutrients and metals occurs in soils with a high content of clay and/or organic matter. Conversely, sandy soils exhibit much lower sorption rates. The same trend holds true for bacterial densities as well (US EPA, 1977). Unfortunately, soils that maximize sorption and bacterial growth also have low and sometimes unacceptable infiltration rates.

TRAPPING/STRAINING

Fine-grained particles eventually become trapped in the void spaces between soil particles as they percolate through the soil.

BACTERIAL REDUCTION

Aerobic bacteria within the soil consume and reduce organic matter. Thelen and Howe (1978) suggests that soil bacteria populations can thrive under porous pavement if the underlying soils get a chance to dry out every few days.

GROUNDWATER DIVERSION

Pollutants which have not been trapped, absorbed or reduced continue to move through the soil profile and into groundwater. This is not a desirable removal method, as it could lead to the contamination of drinking water supplies. Limited studies to date suggest that migration of urban stormwater pollutants through soils is normally not rapid nor deep (Nightingale, 1987; US EPA, 1983; OWML, 1983), except for extremely soluble pollutants such as nitrate or chloride.

Estimates of Porous Pavement Pollutant Removal Efficiency

Two long-term monitoring studies have been conducted in the Washington area on partial exfiltration systems by the OWML (1986b, 1983) in suburban Maryland and Virginia. In both cases, the pollutant export over a series of storms was monitored at a terminal underdrain, and compared to pollutant loads in the runoff from adjacent conventional pavement. Both partial exfiltration sites exhibited similar and quite high removal capabilities. Mass removal of solids was 85% at the Prince William County, Virginia site and 95% at the site in Rockville, Maryland. Approximately 65% of the total phosphorus and 75-85% of the total nitrogen load was removed at both sites. Removal of trace metals, such as zinc and lead, at the Rockville site approached 98%, and over 80% of the COD load was effectively removed (Table 7.2).

In some cases, increased export of the inorganic ions (such as Ca, Mg, K, and Na) has been observed from porous pavement, presumably from the dissolution or leaching of asphalt or stone aggregate (Gburek and Urban, 1980). However, these ions do not represent water quality problems.

Table 7.2: Pollutant Removal Rates Reported at Porous Pavement Sites (Partial Exfiltration Systems)

POLLUTANT	LONG TERM REMOVAL RATE (%)	
	Rockville, Md. Site	Prince William, Va. Site
Sediment	95	82
Total Phosphorus	65	65
Total Nitrogen	85	80
Chemical Oxygen Demand	82	-
Zinc	99	-
Lead	98	-

Sources: OWML, 1986b; MWCOG, 1983b

DESIGN TIPS FOR ENHANCING POLLUTANT REMOVAL

Degree of Exfiltration

The pollutant removal performance of porous pavement depends, to a great extent, on how much of the annual runoff volume is exfiltrated into the soil. Runoff which is not exfiltrated (i.e., collected by an underdrain and routed out of the stone reservoir) receives little effective treatment. For example, OWML (1986b) reported only minor improvement in nutrient concentrations in runoff measured at a terminal underdrain at the Prince William County, Virginia site. Thus, the pollutant removal capability of porous pavement appears to be limited by how much runoff "bypasses" the underlying soil. Only a minor amount of runoff is bypassed in full and partial exfiltration systems for most storms, which contributes to their very high annual removal efficiencies shown in Table 7.2. However, water quality exfiltration systems only exfiltrate approximately 50-75% of the average annual runoff they receive. As a result, removal capability is somewhat reduced. Estimated removal rates for water quality exfiltration systems under various first flush sizing rules are given in Table 5.1.

Surface Area

The more soil surface area available for exfiltration and pollutant adsorption, the better the pollutant removal performance of the structure will be. Thus, a shallow stone reservoir with a large bottom area will normally perform better than a deep one with a smaller bottom area.

Maximum Draining Time

The stone reservoir should be designed to completely drain within a maximum of three days after the maximum design storm event. This allows the underlying soils to dry out and maintain aerobic conditions that favor beneficial bacteria. In addition, this ensures that the stone reservoir will be empty in time for the next storm. Appropriate design techniques are discussed in Md WRA (1984).

If porous pavement is constructed over soils with marginal infiltration capacity (e.g., loams and silt loams), it is prudent to reduce the depth of the stone reservoir (and increase the bottom surface area) so it completely drains within two days. Thus, in the event that the soil infiltration rate turns out to have been overestimated, or diminishes over time because of clogging, the facility will still drain adequately.

Minimum Draining Time

Moderate to poor pollutant removal has been reported for partial exfiltration systems that hold water less than six hours (MWCOG, 1983b). This problem may occur if perforated underdrains are very closely spaced and become too efficient at draining the stone reservoir. The placement of underdrains near the top of the stone reservoir, rather than the bottom, should help to alleviate this problem.

Routine Vacuum Sweeping

The surface of the porous pavement should be vacuum swept at least four times a year, followed immediately by high pressure jet hosing, to keep the asphalt pores free from clogging. Although numerous NURP vacuum sweeping demonstration projects failed to show any great removal of fine-grained pollutants by this method (US EPA, 1983), they did show that sweeping is effective in removing quantities of coarse-grained sediments that are likely to clog porous pavement.

PHYSICAL SUITABILITY AT THE SITE LEVEL

Before porous pavement is constructed, the site should be carefully evaluated to be certain that it is feasible and practical to rely on exfiltration to dispose of runoff. The following factors need to be examined early in the site-planning stage to adequately screen the site. More detailed guidance on porous pavement feasibility can be found in Md WRA (1984) and Thelen and Howe (1978).

Soils

Porous pavement is not feasible for sites with soil infiltration rates of less than 0.27 inches per hour (D soils), or any soil with a clay content greater than 30%. C soils (silt loams and sandy clay loams) provide marginal infiltration rates, and should probably only be considered for partial exfiltration systems (see Table 7.3). Soils that have a combined silt/clay percentage of over 40% by weight are susceptible to frost-heave, and are not good candidates for porous pavement applications. No matter what soil is present, the stone subgrade must extend below the frost line. Also, porous pavement should never be constructed over fill soils, which often form an unstable subgrade, and are prone to slope failure.

If the soils at a site pass these tests, a series of soil cores or trenches should be taken at the site, to a depth at least four feet below the anticipated level of the bottom of the stone reservoir. These should be examined for evidence of any impermeable soil strata that might impede infiltration, such as localized clay lenses, hardpans, or fragipans. The presence of such layers does not necessarily preclude porous pavement applications as long as the stone reservoir penetrates them completely.

Slope

Porous pavement is not recommended on sites with a slope greater than 5%.

Depth to Bedrock

At least two feet of clearance (and preferably four) are needed between the bottom of the stone reservoir and the bedrock level (Md WRA, 1984). This data can be obtained from local soil maps and should always be confirmed by several test soil borings.

Depth to Seasonally High Water Table

A minimum of two to four feet of clearance is needed from the bottom of the stone reservoir and the seasonally high water table. This can be determined by soil borings taken during wet weather.

Expected Traffic Intensity

Porous pavement is generally only used for parking lots and lightly used access roads. Guidelines for determining the corresponding thickness are provided in NVPDC (1987) and Thelen and Howe (1978). If a portion of the parking lot is expected to receive moderate or heavy traffic use, it can be conventionally paved and sloped to drain to a porous pavement area.

Proximity of Wells and Foundations

Porous pavement should be located at least 100 feet away from a drinking water well to minimize the possibility of groundwater contamination, and should be situated at least 10 feet down-gradient from building foundations, and 100 feet up-gradient.

Maximum Depth of Reservoir

To insure that the stone reservoir completely drains in 72 hours, it may be necessary to limit the depth of the stone reservoir if underlying soils have relatively low exfiltration rates. These limits are shown for various soil textures in Table 7.3

Table 7.3: Soil Limitations For Porous Pavement

SOIL TYPE	MINIMUM INFILTRATION RATE (fc--inches/hr)	SCS SOIL[1] GROUP	MAXIMUM DEPTH OF[2] STORAGE* (inches) 48 hrs	72 hrs
Sand	8.27	A	992	595
Loamy Sand	2.41	A	290	174
Sandy Loam	1.02	B	122	183
Loam	0.52	B	62	93
Silt Loam	0.27	C	32	49

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[1] Sandy Clay Loams, Clay Loams, Silty Clay Loams, Sandy Clay, Silty Clay,and Clay Soils are not included as these soil types are all NOT FEASIBLE for infiltration basins.

[2] Maximum Depth of stone reservoir that can drain completely within 48 or 72 hours after a storm, given the soil infiltration rate.

Watershed Size

Md WRA (1984) suggests that porous pavement be restricted to sites between 1/4 acre and ten acres in size. This guideline appears to reflect the fact that other BMPs are more practical and cost-effective outside of this range.

Miscellaneous Factors

Porous pavement is not recommended in areas where wind erosion is expected to supply large quantities of sediment from adjacent barren areas. Also, clogging is likely to be problem if earth-moving equipment or dump-trucks are expected to park at the lot, or if many vehicles are being serviced in or near the lot.

POROUS PAVEMENT COSTS

Predicting Porous Pavement Costs

Porous pavement costs should only be considered as the incremental, or extra costs, incurred over and above the cost of installing a normal conventional parking lot. A preliminary cost estimate can be prepared for a site using the average in-place unit costs combined with the basic geometry of the site. Table 7.4 provides unit cost data for common porous pavement construction components, and was obtained from a survey of over 60 construction bid or bonding estimates prepared by both the public and private sectors in the Washington metropolitan area between 1983 and 1985. An example cost estimate for a hypothetical porous pavement application is provided in Example 7-1. The incremental costs associated with various components of porous pavement are detailed below, and are listed in decreasing order of approximate cost.

Table 7.4: Unit Costs For Porous Pavement Construction Components

ITEM	UNITS[1]	AVERAGE IN-PLACE UNIT COST[2]	TYPICAL RANGE
Common Excavation	cy	2.82	2.00-5.00
Clear and Grub	ac	2800.00	1,500-3,500
Seed/Mulch	sy	0.58	0.25-1.00
Rip-Rap	sy	38.00	25.00-55.00
Select Fill	cy	3.97	3.00-5.50
Silt Fence	lf	4.11	2.00-5.00
Gabions	cy	114.00	***
Filter Cloth	sy	2.71	2.00-5.00
PVC Pipe			
6 inch	lf	10.00	8.00-12.00
8 inch	lf	10.50	***
10 inch	lf	15.00	***
Stone Fill (1-2")	cy	22.50	15.00-25.00
Clean Washed Sand	cy	14.00	***
Pea Gravel	cy	7.50	***
Stone Tamping	cy	2.00	***
Observation Well	lf	150.00	25.00-400.00
Sediment Control	lf		1,000-8,000

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[1] Unit cost data derived from MWCOG (1983a) and supplemented by 45 itemized SWM construction bids or bonding estimates analyzed in the Washington, D.C. area, 1983-1986. Items for which less than five independent estmates were available are denoted by ***. Material costs may vary among jurisdictions and regionally.

[2] cy=cubic yard, sy=square yard, ac=acre, lf=linear foot.

ADDITIONAL DEPTH OF STONE RESERVOIR

This refers to the difference between the normal design depth of stone aggregate under conventional pavement and the design depth of the stone reservoir needed for porous pavement. The additional excavation and stone backfill runs about 80-90 cents per cubic foot. The depth of the stone reservoir varies from site to site, depending on the volume of the design storm controlled, degree of off-site drainage (if any), and the soil infiltration rate. The depth of the stone aggregate required for conventional pavement is also variable, but tends to average about 6 feet in most low intensity parking situations. At most sites, the extra excavation and stone fill needed for the deeper stone reservoir is the single largest cost component for porous pavement.

EXTRA COST FOR POROUS ASPHALT

Normal asphalt typically runs about $1.10-1.20 per square foot. Reported prices for porous asphalt typically run about 10-15% higher ($1.30-1.40 per square foot) although recently reported unit costs from some jobs have been as high as $2.00 per square foot. The price differential reflects the extra costs involved in procuring, producing, transporting and rolling the porous asphalt, and probably will narrow as the demand for this relatively new product increases.

EXTRA COST FOR FILTER FABRIC

Filter fabric, or similar geo-textiles, are needed to line the bottom and sides of the stone reservoir to prevent lateral or upward movement of soil into the stone reservoir. Although the unit price of filter fabric is not great (approximately $0.30 per square foot), the large surface area that must be lined make it a high cost component.

EXTRA COST FOR SEDIMENT AND EROSION CONTROL

Greater efforts must be made in sediment and erosion control as an insurance policy against premature clogging. Sediment and erosion controls must keep sediment completely out of the planned porous pavement area, rather than the normal practice of merely keeping sediment from leaving the construction site. Normally, this entails constructing a diversion berm around the perimeter of the pavement area, or less preferably, the installation of an extensive filtering system (e.g., silt fences or Austin triangles at the end of a sodded waterway). A sediment basin will still be needed elsewhere at the site to trap sediment. As a result, the common cost saving practice of converting a sediment basin into a stormwater pond after construction is completed cannot be used. Although sediment and erosion control costs vary from site to site, a lump sum cost of between $2000 and $5000 per acre controlled is often assigned in many area jobs (Md WRA, 1986b). It is probably reasonable to assume that an extra $1000 will need to be specifically targeted for sediment and erosion control measures to protect the pavement area.

EXTRA COST FOR PRE-TREATMENT

A 20 foot wide grass buffer is required around the perimeter of the pavement if adjacent areas will contribute runoff to the pavement area. Unit prices for topsoil, mulch and seed average about 58 cents per square yard. If the porous pavement accepts runoff from off-site areas, more expensive pre-treatment facilities such as pre-cast drop boxes, oil/grit chambers or sand filters will be needed.

EXTRA COST FOR UNDERDRAINAGE

For partial exfiltration systems, a network of perforated underdrains must be installed. The unit price depends on the diameter and material of the pipe selected. Underdrainage may not always be considered an extra cost; larger, conventionally paved parking lots may also require the installation of an underground pipe network.

EXTRA CONTINGENCY COST

After all in-place unit costs are estimated, it is customary for a contractor to add an additional 25% to the total cost to cover contingencies, such as job design, inspection, oversight and administration. Because more extensive soil testing, drainage surveys, inspection, and higher workmanship is required for porous pavement, it may more realistic to add at least 10% more to the normal contingency rate for porous pavement jobs.

Potential Savings Associated with Porous Pavement

Balanced against the extra costs are the following savings that might result from porous pavement:

REDUCTION/ELIMINATION OF CURB AND GUTTERS

With the exception of a raised curb on the down-slope portion of a porous pavement site, the standard curb and gutter system needed for most conventional parking lots can be reduced in size or eliminated altogether.

REDUCED LAND CONSUMPTION

Cost savings can be realized since porous pavement conserves the amount of developable land on a property. Additional land at the site is not needed for stormwater management purposes, and is then available for more economic purposes. Also, porous pavement is an attractive option when the size of the development, or lack of available space, limit the use of other BMPs.

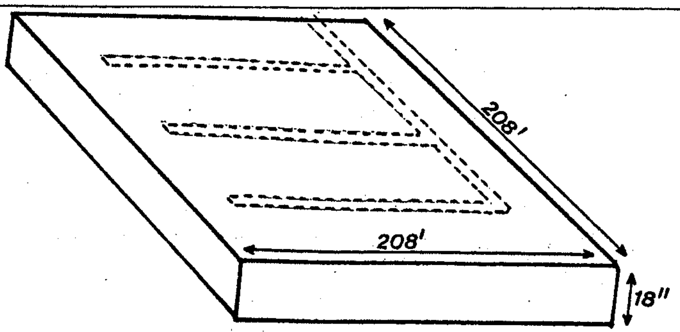
REDUCED RUNOFF STORAGE REQUIREMENTS

By using porous pavement rather than impermeable conventional pavement, the developer can reduce the runoff storage requirement for the site, if permitted by the local reviewing authority.

Example 7-1: UNIT COST ESTIMATION FOR A HYPOTHETICAL POROUS PAVEMENT PARKING LOT.

Estimate the cost of constructing a one acre partial exfiltration porous pavement parking lot which accepts runoff from an adjacent one acre parking lot of conventional pavement. Preliminary design calculations indicate that the stone reservoir will be 18 inches deep, compared to 6 inches for the adjacent conventional paving. Overflow pipes are spaced as shown in the figure below.

EXAMPLE 7-1
continued



1. EXCAVATION/STONE FILL FOR EXTRA DEPTH OF STONE RESERVOIR:
   18-6 = 12 inches or 1 ft extra reservoir
   (43560 sf)(1 ft) = 43560 cf or 1613 cy

   Excavation @ $2.82/cy * 1613 cy = $4,549
   Stone Fill @ $22.50/cy * 1613 cy = $36,293

2. EXTRA COST FOR FILTER CLOTH:
   Bottom = 43,560 sf
   Sides = (4)(208)(1.5) = 1248 sf
   \+ 5% allowance for overlap = 47,048 sf or 5228 sy

   Filter Cloth @ $2.71/sy * 5228 sy = $14,167

3. EXTRA COST FOR POROUS ASPHALT:
   Assume 15% price differential for porous pavement,
   and $10.50 unit price for conventional pavement:
   Extra Cost = (0.15)($10.50/sy) = $1.58 sy
   43,560 sf = 4840 sy

   Porous Asphalt @ $1.58/sy * 4840 sy = $7,647

4. EXTRA COST FOR OVERFLOW PIPES:
   Assume three (3), 6" PVC pipes spaced 50' apart and
   100' in length, with a 8" lateral PVC pipe, 150' long:

   300' of 6" PVC @ $10.00/lf = $3,000
   150' of 8" PVC @ $10.50/lf = $1,575

5. EXTRA COST FOR GRASS BUFFER: (none in this example) 0

6. EXTRA COST FOR SEDIMENT AND EROSION CONTROL:
   Assume lump sum of $1000.00 for special measures. $1,000

7. SUB TOTAL $68,231

8. EXTRA CONTINGENCY COSTS
   @ 0.10 of SUB-TOTAL = 6,823

9. GRAND TOTAL $75,054

NOTE: Any cost savings resulting from reduced requirements for curbs and gutters, pipe-sizing, or SWM land consumption should be subtracted from the grand total before comparing it to other BMP alternatives.

Due to all the complexities in the design and economics of porous pavement, it is not easy to compare it with other competing BMP alternatives. Rather crude cost comparisons contained in Wiegand et al. (1986) and Schueler et al. (1985) suggest that both partial and complete exfiltration systems can be extremely cost-effective when applied to smaller parking lots (10 acres or less), with no off-site runoff contribution. Both systems are reasonably cost-effective if the off-site runoff contribution is less than or equal to that of the porous pavement area. If the off-site runoff contribution is greater than the pavement area, and/or the pavement area is greater than ten acres, porous pavement is not usually very competitive in relation to other BMP options. This is due to the fact that economies of scale (which are very pronounced for ponds) are not particularly evident in porous pavement applications.

CONSTRUCTION SPECIFICATIONS AND MAINTENANCE REQUIREMENTS

Proper construction and routine maintenance are extremely important for porous pavement. If installed properly, porous pavement should last as long as conventional pavement. However, a substantial number of recent projects have failed shortly after being built, primarily due to poor construction practices, inadequate field testing or lack of sediment control. Appropriate preventative measures are discussed below (Maryland WRA, 1986b; City of Rockville, Maryland, 1984a; Diniz, 1980; Thelen and Howe, 1978).

Construction Specifications

Table 7.5 summarizes the construction specifications for preparing the stone reservoir and porous asphalt layer. As can be seen, porous asphalt has special requirements during each phase of installation: mixing, transport, laying and rolling. Similar care needs to be taken during the preparation of the stone reservoir. Rather than provide detailed step by step guidance on the entire construction procedure (which is well summarized in Md WRA, 1986b, 1984; Diniz, 1980), the summary below emphasizes those practices that can prevent premature clogging during the construction phase:

1. Before the entire development site is graded, the planned area for the porous pavement should be roped off to prevent heavy equipment from compacting the underlying soils.

2. Diversion berms should be placed around the perimeter of the porous pavement to keep runoff and sediment <u>completely</u> away from the site both before and during construction.

3. Excavation of the sub-grade should be performed by earthmoving equipment with tracks or over-sized tires. Normal rubber tires should be avoided since they compact the subsoil and reduce its infiltration capabilities.

4. After excavation is completed, the bottom and sides of the stone reservoir should be lined with filter fabric to prevent upward piping of underlying soils. The fabric should be placed flush with a generous overlap between rolls.

5. Clean, washed 1-2 inch stone aggregate should be placed in the excavated reservoir in lifts, and <u>lightly</u> compacted with plate compactors to form the base course. Unwashed stone has enough associated sediment to pose a clear risk of clogging at the soil/filter cloth interface.

Table 7.5: Construction Specifications For Porous Pavement

STABILIZATION

TO PRECLUDE PREMATURE CLOGGING AND/OR FAILURE OF THIS PRACTICE POROUS PAVING STRUCTU SHALL NOT BE PLACED INTO SERVICE UNTIL ALL OF THE SURFACE DRAINAGE AREAS CONTRIBUTING THE PAVEMENT HAVE BEEN EFFECTIVELY STABILIZED IN ACCORDANCE WITH THE MARYLAND STANDA AND SPECIFICATIONS FOR SOIL EROSION AND SEDIMENT CONTROL FOR DISTURBED AREAS.

FILTRATION

WHEN OVERLAND FLOW FROM ADJACENT AREAS IS DIRECTED TOWARD THE PARKING LOT, A DES VEGETATIVE FILTER STRIP WILL BE REQUIRED TO PROTECT THE PAVED AREA. SEE SECTION 3.7 VEGETATIVE FILTER STRIP DESIGN SPECIFICATIONS.

SUBGRADE PREPARATION

1. ALTER AND REFINE THE GRADES AS NECESSARY TO BRING SUBGRADE TO REQUIRED GRADES SECTIONS AS SHOWN IN THE DRAWINGS.
2. THE TYPE OF EQUIPMENT USED IN SUBGRADE PREPARATION CONSTRUCTION SHALL NOT CAUSE UN SUBGRADE COMPACTION. (USE TRACKED EQUIPMENT OR OVERSIZED RUBBER TIRE EQUIPMENT - NOT USE STANDARD RUBBER TIRED EQUIPMENT.) TRAFFIC OVER SUBGRADE SHALL BE KEPT A MINIMUM. WHERE FILL IS REQUIRED, IT SHALL BE COMPACTED TO A DENSITY EQUAL TO UNDISTURBED SUBGRADE, AND INHERENT SOFT SPOTS CORRECTED.

STONE BASE COURSE

1. ALL STONE USED SHALL BE CLEAR, WASHED, CRUSHED STONE, MEETING LOCAL HIGHWAY DEPARTM SPECIFICATIONS.
2. STONE SHALL BE OF TWO SIZES: THE RESERVOIR BASE COURSE SHALL BE TO DEPTH AS NOTED DRAWINGS OF CRUSHED STONE (MAXIMUM OF 2", MINIMUM OF 1"), AND A 2-INCH DEEP TOP COU OF 1/2" STONE (MAXIMUM OF 5/8", MINIMUM OF 3/8").
3. STONE BASE COURSE SHALL BE LAID OVER A DRY SUBGRADE COVERED WITH ENGINEERING FIL FABRIC SUCH AS MIRAFI #14N OR EQUAL TO A DEPTH SHOWN IN DRAWINGS, IN LIFTS TO NATURALLY COMPACTED. THE STONE BASE COURSE SHALL BE COMPACTED LIGHTLY. KEEP THE B COURSE CLEAN FROM DEBRIS, AND SEDIMENT.

POROUS ASPHALTIC CONCRETE SURFACE COURSE

1. THE SURFACE COURSE SHALL BE LAID DIRECTLY OVER THE 1/2" STONE BASE COURSE AND SHALL LAID IN ONE LIFT.
2. THE LAYING TEMPERATURE SHALL BE BETWEEN 240 AND 260, WITH MINIMUM AIR TEMPERATURE O 50F, TO MAKE SURE THAT THE SURFACE DOES NOT COOL PRIOR TO COMPACTION.
3. COMPACTION OF SURFACE COURSE SHALL BE DONE WHILE THE SURFACE IS COOL ENOUGH TO RES A 10-TON ROLLER. ONE OR TWO PASSES BY THE ROLLER IS ALL THAT IS REQUIRED FOR PRO COMPACTION. MORE ROLLING COULD CAUSE A REDUCTION IN THE SURFACE COURSE POROSITY.
4. MIXING PLANT SHALL CERTIFY THE AGGREGATE MIX AND ABRASION LOSS FACTOR AND THE ASPH CONTENT IN THE MIX. THE ASPHALTIC MIX SHALL BE TESTED FOR ITS RESISTANCE TO STRIPP BY WATER USING ASTM D 1664. IF THE ESTIMATED COATING AREA IS NOT ABOVE 95 PERCE ANTI-STRIPPING AGENTS SHALL BE ADDED TO THE ASPHALT.
5. TRANSPORTING OF MIX TO SITE SHALL BE IN CLEAN VEHICLE WITH SMOOTH DUMP BEDS THAT H BEEN SPRAYED WITH A NON-PETROLEUM RELEASE AGENT. THE MIX SHALL BE COVERED DUR TRANSPORTATION TO CONTROL COOLING.
6. MIX OF ASPHALTIC CONCRETE SHALL BE 5.75 TO 6 PERCENT OF WEIGHT OF DRY AGGREGATE.
7. ASPHALTIC GRADE SHALL MEET AASHTO SPECIFICATION M-20 FOR 85 TO 100 PENETRATION R ASPHALT AS A BINDER IN THE NORTHERN UNITED STATES, 65 TO 80 IN THE MIDDLE STATES, 50 TO 65 IN THE SOUTH.
8. AGGREGATE GRADING SHALL BE AS SPECIFIED IN TABLE 3-3.

PROTECTION

AFTER FINAL ROLLING, NO VEHICULAR TRAFFIC OF ANY KIND SHALL BE PERMITTED ON PAVEMENT UNTIL COOLING AND HARDENING HAS TAKEN PLACE, IN NO CASE SHOULD THE TIME BE L THAN 6 HOURS (PREFERABLY A DAY OR TWO).

WORKMANSHIP

1. WORK SHALL BE DONE EXPERTLY THROUGHOUT AND WITHOUT STAINING OR INJURY TO OTHER PERMANENT WORK.

2. MAKE TRANSITION BETWEEN EXISTING AND NEW PAVING WORK NEAT AND FLUSH.

3. FINISHED PAVING SHALL BE EVEN, WITHOUT POCKETS, AND GRADED TO ELEVATIONS SHOWN. POROUS PAVING WITH RESERVOIRS BELOW FROST LINE CAN AND SHOULD BE AS FLAT AS POSSIBLE.

MAINTENANCE

THE SURFACE OF POROUS PAVEMENT ESPECIALLY WHERE WATER IS CONCENTRATED MUST BE CLEANED REGULARLY TO AVOID ITS BECOMING CLOGGED BY FINE MATERIAL. THIS CLEANING IS BEST ACCOMPLISHED THROUGH USE OF A VACUUM CLEANING STREET SWEEPER. OUTSIDE OF REGULAR CLEANING, POROUS PAVEMENT REQUIRES NO MORE MAINTENANCE THAN CONVENTIONAL PAVEMENT. IN TIMES OF HEAVY SNOWFALL IT MUST BE RECOGNIZED THAT APPLICATION OF ABRAISIVE MATERIAL SHOULD BE CLOSELY MONITORED TO AVOID CLOGGING PROBLEMS ONCE THE SNOW AND ICE MELTED. NO METHOD OF MAINTENANCE HAS BEEN SATISFACTORY ON FULLY CLOGGED PAVEMENTS, AND ONLY A SUPERFICIALLY CLOGGED SECTION SHOWING A WATER INFILTRATION RATE OF 0.1 INCHES PER SECOND COMPARED TO A NORMAL WATER PENETRATION OF 0.38 INCHES PER SECOND CAN BE RESTORED TO NORMAL OPERATION. THE BEST METHOD FOR CLEANING IS BRUSH AND VACUUM SWEEPING FOLLOWED BY HIGH PRESSURE WATER WASHING OF THE PAVEMENT. VACUUM CLEANING ALONG, ONCE THE PAVEMENT IS CLOGGED, HAS BEEN FOUND INEFFECTIVE. THE OILS IN THE ASPHALT BIND DIRT, AND ONLY AN ABRADING AND WASHING TECHNIQUE CAN BE EFFECTIVE IN THE REMOVAL OF SUCH DIRT. CLOGGING TO A DEPTH OF 0.5 INCH IS SUFFICIENT TO PREVENT WATER PENETRATION. FOR CLOGGED PAVEMENT, DRILLING OF ONE QUARTER INCH HOLES EACH SQUARE FOOT IS RECOMMENDED TO RESTORE ORIGINAL DRAINAGE CAPACITY.

TRAFFIC CONTROL

EXPERIENCE HAS SHOWN THE NEED FOR CLOSE CONTROL OF CONTRACTOR VEHICLES ON NEWLY INSTALLED AREAS OF POROUS PAVEMENT. DAMAGE TO PAVEMENT POROSITY RESULTS CHIEFLY FROM ABUSE DURING THE EARLY LIFE OF THE PAVEMENT. NORMALLY, PAVING IS DONE WHILE HEAVY CONSTRUCTION OR EARTH MOVING IS CONTINUING IN AN AREA. THE PAVEMENT IS THUS SUBJECTED TO MUD AND DIRT FROM CONTRACTOR VEHICLES FOR UP TO SEVERAL MONTHS, AND THE CONTINUAL PASSAGE OF THESE VEHICLES COMPACTS THE DIRT INTO THE PORES. ONLY IF CAKED MUD IS CLEANED FROM VEHICLE WHEELS AND THE PAVEMENT IS CLEANED DAILY BY SWEEPING AND HIGH-PRESSURE WATER WASHING CAN POROSITY BE RETAINED. CLOGGING CAN BE FURTHER MINIMIZED BY PROPER USE OF CURBING TO PREVENT SURROUNDING SOILS FROM WASHING ONTO THE PAVEMENT SURFACE.

TYPE AND QUALITY OF AGGREGATE

THE AGGREGATES SELECTED FOR POROUS PAVEMENT CONSTRUCTION SHOULD MEET REQUIREMENTS OF THE STANDARD SPECIFICATION FOR "CRUSHED STONE, CRUSHED SLAG AND CRUSHED GRAVEL FOR DRY-OR-WATER-BOUND MACADAM BASE AND SURFACE COURSED OF PAVEMENTS," ASTM D693-77, WITH TWO EXCEPTIONS, FIRST, THE GRADATION TEST MUST BE OF THE OPEN GRADED TYPE DESCRIBED HERE. SECOND, A SOUNDNESS TEST IS REQUIRED, AS SPECIFIED IN ASTM D 692-79, COURSE AGGREGATE FOR BITUMINOUS PAVING MIXTURES, TO DETERMINE IF THE AGGREGATE IS SUSCEPTIBLE TO DISINTEGRATION BY WATER.

ASPHALT CEMENT GRADE IN MIX

THE SUGGESTED VISCOSITY GRADE OF ASPHALT CEMENT TO BE USED IS AS-20 AASHTO M-226-73 I. THIS GRADE IS TO CONSIDERED A TENTATIVE STARTING POINT BECAUSE TEST RESULTS OBTAINED FROM THE DESIGN PROCESS MAY INDICATE AN ADVANTAGE OR A NECESSITY TO ALTER THE ASPHALT GRADE.

MIXING TEMPERATURE

TO ENSURE THE INDIVIDUAL AGGREGATE PARTICLES ARE COMPLETELY SURROUNDED BY ASPHALT, AND THAT THE ASPHALT IS TIGHTLY BOUND TO EACH PARTICLE, TEMPERATURE OF MIXING AT THE HOT MIX PLANT MUST BE RIGIDLY CONTROLLED. TO LOW A MIXING TEMPERATURE WILL RESULT IN INADEQUATE ASPHALT BINDING AND COVERAGE OF THE AGGREGATE, WHILE TOO HIGH A MIXING TEMPERATURE WILL ALLOW ASPHALT TO DRAIN FROM THE MIX, RESULTING IN A LOWER ASPHALT CONTENT AND DECREASED STRENGTH. SUITABLE MIXING TEMPERATURES RANGE FROM 230 TO 260 DEGREES FARENHEIT, BUT THE LOWER END OF THAT RANGE (230 TO 240F) IS RECOMMENDED.

ASPHALTIC CONTENT IN MIX

FOR ROAD PAVING DURABILITY AND TO PREVENT TOO RAPID HARDENING OF THE ASPHALT, IT IS DESIRABLE TO HAVE THE HIGHEST ASPHALT CONTENT POSSIBLE IN THE MIX. TOO MUCH ASPHALT WOULD SEPARATE OUR UNDER TRAFFIC, SO THAT MAXIMUM ASPHALT CONTENT IS GENERALLY LIMITED BY THAT FACTOR. EXPERIENCE HAS SHOWN THAT 5.5 PERCENT BY WEIGHT IS THE MINIMUM RECOMMENDED ASPHALT CONTENT. ASPHALT CONTENT SHOULD BE DETERMINED ACCORDING TO THE TESTING PROCEDURE RECOMMENDED IN FEDERAL HIGHWAY ADMINISTRATION REPORT NO. FHWA-RD-74-2, ALREADY CITED. THE MARSHALL DESIGN METHOD FOR DETERMINING MIX CONTENT IS NOT RECOMMENDED. USING A 5.5 PERCENT ASPHALT CONTENT AND THE ASPHALT INSTITUTE'S RECOMMENDED 4-INCH MINIMUM SURFACE COURSE, A 0.6 INCH RAINFALL RESERVOIR CAPACITY IS OBTAINED WITH AN INFILTRATION RATE OF 176 INCHES PER HOUR.

Table 7.5 (continued)

OPEN-GRADED ASPHALT CONCRETE FORMULATION

MATERIAL	PASSES THRU SCREEN:	WEIGHT (%)	VOLUME (%)	---- PROBABLE PARTICLE DATA		-----------
				width, mm	weight,g	No. Per 100g of asphalt concrete
AGGREGATE	1/2	2.8	2.2	10.7	1.667	1.7
	3/8	59.6	46.3	8.0	0.697	85.5
	#4	17.0	13.3	4.0	0.087	195.4
subtotal coarse aggregate:		79.4	61.8			282.6
	#8	2.8	2.2	2.0	0.0109	255.6
	#16	10.4	8.0	1.0	0.00136	7647.0
	#200	1.9	1.5	0.06	0.000294	6462.0
ASPHALT		5.5	10.5			
AIR		0.0	16.0			
		100.0	100.0			

SOURCE: City of Rockville (1984a).

6. A one-inch deep layer of 3/8- to 5/8-inch stone is placed over the base course, and *manually* graded to plan specifications.

7. The porous asphalt layer is then added, when the air temperature is above 50 degrees F and the laying temperature is between 230-260 degrees F. Failure to follow these guidelines can lead to premature hardening of the asphalt and subsequent loss of infiltration capacity.

8. Rolling can begin when the asphalt is cool enough to withstand a ten ton roller. Normally, only one or two passes of the roller are necessary. More frequent rolling can reduce the infiltration capabilities of the open-graded asphalt mix.

9. After rolling is complete, all traffic should be kept out of the porous pavement area for a minimum of one day to allow proper hardening.

10. *Post-construction sediment control is critical.* The majority of porous pavement failures occur in the first few weeks and months after the asphalt has been rolled, usually from clogging caused by adjacent erosion or sediment tracked in from elsewhere on the site by construction vehicles. Therefore, it is very important that 1) S&E practices be inspected to make sure they still work, 2) the vegetated buffer strips are immediately established, 3) reinforced silt fences or Austin triangles are placed between the buffer and pavement to prevent sediment entry until the buffer is well established, 4) signs are posted and construction personnel advised not to enter the parking lot with muddy tires, and 5) if such traffic cannot be prohibited, a temporary stone construction entrance should be installed.

Routine Maintenance

The following routine maintenance tasks should be a legally binding element of the property deed:

VACUUM SWEEPING

The porous pavement surface should be vacuum swept at least four times per year, followed by high-pressure jet hosing, to keep the asphalt pores open. Several firms in the region now provide this service as part of a regular, relatively low cost contract. Evidence of such a contract should be provided to the inspector before any bonds are released on the job.

INSPECTION

The pavement should be inspected several times in the first few months after construction, and then annually thereafter. Inspections should be conducted after large storms to check for surface ponding that might indicate local or widespread clogging. Also, the condition of the vegetated buffer strips should be examined.

PATCHING

Potholes and cracks can be repaired using conventional, non-porous patching mixes as long as the cumulative area repaired does not exceed 10% of the parking lot area.

RELIEVING SURFACE CLOGGING

Spot clogging of the porous pavement layer can be relieved by drilling half-inch holes through the porous asphalt layer every few feet. In cases where clogging occurs in a low spot in the parking lot, it may be advisable to install a drop inlet to route water into the stone reservoir.

SNOW REMOVAL

Sand or ash should never be applied to porous pavement for snow removal purposes. This site should be posted to that effect. Thelen and Howe (1980) report that snow and ice melt is more rapid on porous pavement than conventional pavement, which suggests that prohibiting these materials may not be a major inconvenience.

Non-Routine Maintenance

The routine maintenance tasks outlined above should prevent or relieve surface clogging in the asphalt layer. A much more serious problem occurs if the subsoil, or the subsoil/filter cloth interface becomes clogged over time. At present, nothing short of complete replacement can correct this condition. It may be advisable to install a backup underdrainage system of capped perforated pipes to convert the pavement into a partial exfiltration system in the event of bottom clogging (particularly if subsoils initially have marginal infiltration capacity).

Total Maintenance Costs

Very limited data is presently available to assess porous pavement maintenance costs. However, it is probably reasonable to assume that routine costs are lower in comparison to ponds, whereas non-routine maintenance costs may run much higher.

ENVIRONMENTAL ATTRIBUTES OF POROUS PAVEMENT

Impacts to the Natural Environment

Both partial and complete porous pavement designs get high marks for protecting downstream aquatic life (if any exists), as they maintain the pre-development water balance at the site, minimize streambank erosion and filter out pollutants. Also, because of the high groundwater recharge associated with porous pavement, perimeter landscape plantings will tend to grow more vigorously than those adjacent to conventional pavement.

One potential negative impact of porous pavement is the risk of groundwater contamination. Long-term studies of pollutant migration in soils underneath various infiltration practices indicate only limited downward migration of pollutants through the soil (Nightingale, 1987; OWML, 1983, Appendix E; US EPA, 1983). Possible exceptions include very soluble pollutants such as nitrate, and chlorides. Also, there is some concern that some toxic chemicals could potentially leach from the asphalt or binder, although the limited lysimeter studies to date have not confirmed this (Gburek and Urban, 1980). The possible risks of groundwater contamination by porous pavement is currently the object of an intensive monitoring survey conducted by the USGS. Until more is known, it is advisable not to site porous pavement near groundwater drinking supplies.

Impacts on the Human Environment

In terms of traffic safety, porous pavement is generally superior to conventional pavement. Thelen and Howe (1978) report that vehicles have better skid resistance and are less susceptible to hydroplaning on porous pavement, in comparison to regular paving. Porous pavement also produces less headlight glare on rainy nights.

RELEVANT DESIGN GUIDANCE

Table 7.7 summarizes some of the design criteria for porous pavement, which are also shown in schematic form in Figure 7.6. In addition, the following references should be consulted for more detailed guidance on the design and installation of porous pavement:

Maryland Water Resources Administration, 1983. Standards and Specifications for Infiltration Practices.

Maryland Water Resources Administration, 1985a. Inspectors Guideline Manual for Stormwater Management Infiltration Practices.

DESIGN SUMMARY: POROUS PAVEMENT

- **SITE EVALUATION:**
  Prior to design, the site should be carefully evaluated to determine whether it is feasible for infiltration. This involves taking at least three soil borings or trenches, to a depth of 4 feet below the anticipated bottom of the stone reservoir. Evidence of the seasonally high water table, bedrock level, fill soils, or localized clay lenses should not be present in the cores. Underlying soils should have a minimum infiltration rate of 0.27 in/hr (for partial exfiltration systems) or 0.52 in/hr (for full exfiltration systems).

- **TRAFFIC INTENSITY:** Porous pavement is generally only feasible for low volume automobile parking areas (0.25 to 10.0 acres in size) and lightly used access roads. Areas within a large parking lot that are expected to receive moderate or heavy traffic intensity, or that will accommodate heavy trucks, can be conventionally paved and then sloped to drain over to an adjacent porous pavement area.

- **DEGREE OF EXFILTRATION:**
  Most porous pavement applications will be of the full or partial exfiltration variety, and should be designed to exfiltrate a minimum of runoff volume equivalent to the first one-half inch of runoff from contributing impervious areas.

- **SLOPE:**
  The slope of porous pavement should not exceed 5% and is best when as flat as possible. If low spots do develop in the parking lot, it may be advisable to install drop inlets to divert runoff into the stone reservoir more quickly.

- **CONSTRUCTION:**
  Probably more than any other BMP, porous pavement requires a high level of construction expertise and workmanship. The construction guidelines presented in Table 7.5 should be followed precisely.

- **MAXIMUM DRAINING TIME:**
  The depth of the stone reservoir should be adjusted so it drains completely within 72 hours. This allows the underlying soils to dry out between storms (improving pollutant removal) and also preserves capacity for the next storm. If the site has marginal soils for infiltration (loams, silt loams), or covers a wide area, it may be prudent to design the reservoir to drain within 48 hours.

- **MINIMUM DRAINING TIME:**
  Care should be taken in spacing the underdrain network in partial exfiltration systems. If perforated underdrains are spaced too close together, runoff may be collected too efficiently to provide the exfiltration needed for high pollutant removal. As a general design rule, a minimum residence time of 12 hours (as determined by the modified TR-20 procedure for infiltration facilities) should be a target for the design event.

- **OBSERVATION WELLS:**
  An observation well, consisting of a well-anchored, vertical perforated PVC pipe with a lockable aboveground cap, should be installed on the downslope end of the porous pavement area to monitor runoff clearance rates. The well should be checked several times in the first few months after construction. Water depth in the well should be measured at 0, 24 and 48 hour intervals after a storm. Clearance rates are calculated by dividing the drop in water level (in) by the time elapsed (hrs) from the end of the storm. A series of clearance rate measurements taken over the years provides a useful tool for tracking any clogging problems within the stone reservoir.

- **POSTING:**
  The porous pavement site should be posted with signs indicating the nature of the surface, and warning against resurfacing the site with conventional pavement, using abrasives (such as sand or ash) for snow removal, or parking of heavy construction equipment.

- **EROSION CONTROL:**
  Sediment must be kept completely away from a porous pavement site before, during and after construction. Diversion berms should be used to divert stormwater and sediment around the planned porous pavement site. Porous pavement construction should never begin until all contributing upland areas have been completely stabilized. Soil excavated during construction should be placed well away from the perimeter of the site to prevent it from washing back into the stone reservoir.

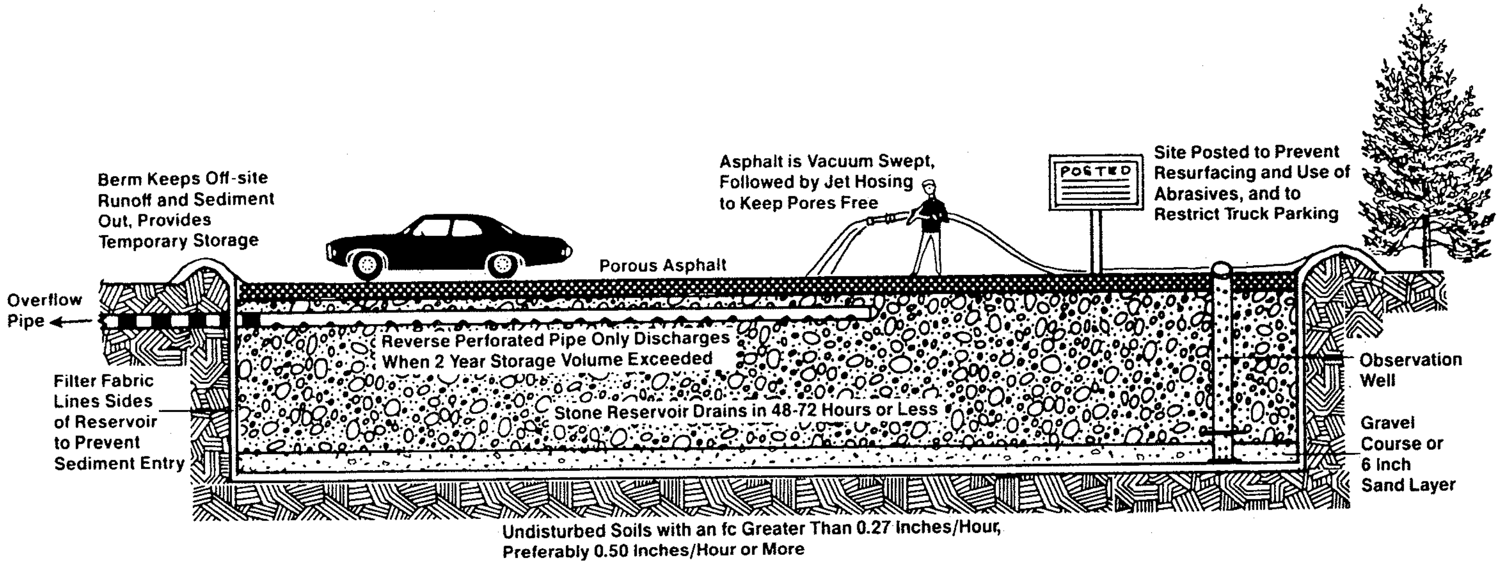
- **PRETREATMENT OF RUNOFF:**
  If the porous pavement site receives runoff from off-site areas, a pretreatment facility should be constructed to remove oil, grit and sediments before they can enter the stone reservoir and possibly clog it. Sand filters, water quality inlets, short trenches or barrel inlets (rooftop runoff only) can be used for this purpose.

- **VACUUM SWEEPING/JET HOSING:**
  The pavement surface should be vacuum swept at least four times per year to remove any grit or sediment trapped in the pores of the open-graded asphalt. This treatment should be immediately followed by high pressure jet hosing to wash off any remaining fine particles. Evidence of a regular service contract for performing this important maintenance activity should be required before any bonds are released on a project.

- **INSPECTIONS:**
  Each site should be inspected annually during wet weather to check the clearance rate of the stone reservoir, and to inspect the condition of buffer strips, pretreatment facilities, and any evidence of surface clogging.

- **CLOGGING/REPAIRS:**
  Potholes, cracks and other pavement defects can be patched with conventional paving mixes, as long as the cumulative total area repaired is less than 10% of the total area. If the regular vacuum sweeping/hosing routine does not relieve surface clogging, half-inch diameter holes can be drilled through the asphalt course into the stone reservoir to facilitate drainage. If the stone reservoir or subsoil becomes clogged, the structure may have to be replaced, unless a backup system of underdrains is provided.

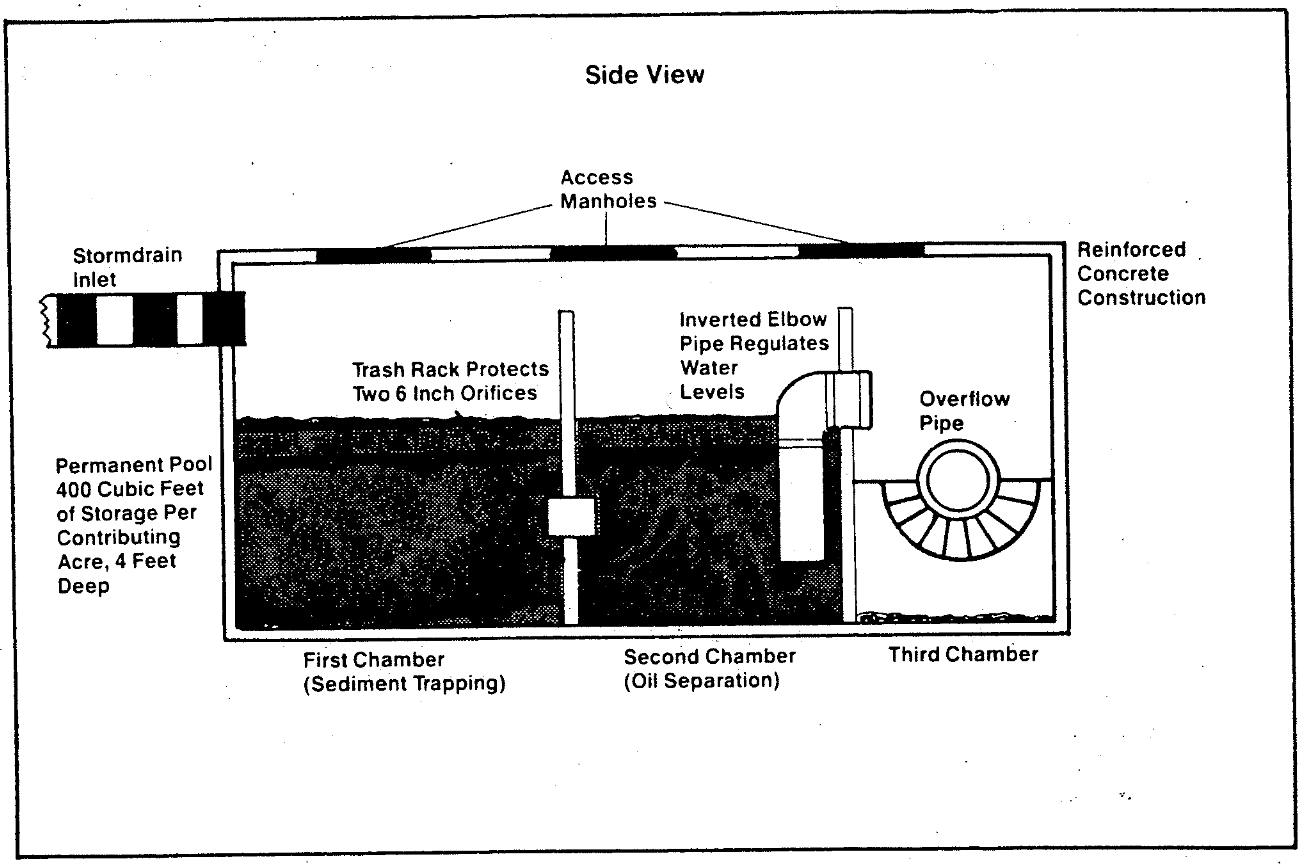
Figure 7.6: Design Schematic for Porous Pavement



CHAPTER 8: WATER QUALITY INLETS

Water quality inlets (also known as oil/grit separators) are designed to remove sediment and hydrocarbon loadings from parking lot runoff before they are conveyed to the storm drain network or to an infiltration BMP. Under current designs, water quality inlets only store a small fraction of the two year design storm volume, and because of their limited capacity, inlets play no role in modifying the post development peak discharge rate. The pollutant removal capability of these inlets has never been monitored in the field. However, since runoff is only briefly retained in the inlets, only moderate removal of coarse sediment, oil/grease, and debris can be expected. Even more limited removal is likely for fine-grained particulate pollutants such as silt, clay and associated trace metals and nutrients. Soluble pollutants probably pass through inlets without modification. Water quality inlets typically serve parking lots one acre or less in size, and are particularly appropriate for sites that are expected to receive a great deal of vehicular traffic or petroleum inputs (e.g, gas stations, roads, loading areas). Installation costs of standard sized water quality inlets are on the order of $5000-15,000. Routine maintenance costs are high since the inlets must be cleaned out at least twice a year to permanently dispose of trapped pollutants and to ensure proper inlet function.

Figure 8.1: Schematic of a Water Quality Inlet, Montgomery County, MD. Three Chamber Design



Advantages of the water quality inlets lie in their unobtrusiveness, compatibility with the storm drain network, easy access, and capability to pretreat runoff before it enters infiltration BMPs. Disadvantages include their limited stormwater and pollutant removal capabilities, the need for frequent clean-outs (which cannot always be assured), and possible difficulties in disposing of accumulated sediments.

INLET DESIGN VARIATIONS

Montgomery County Design

A typical three chamber design for a water quality inlet, developed in Montgomery County, Maryland (MCDEP, 1984b), is shown in Figure 8.1. Basically, the inlet is a long rectangular concrete chamber connected to the storm drain system. Runoff passes through three chambers that are specially modified to separate out sediment, grit and oil before exiting through a storm drain pipe.

The first chamber in the inlet contains a permanent pool of water that is three to four feet deep, and is connected to the second chamber by a pair of well-screened six inch holes. The first chamber is used for gravity settling of grit and sediments, and can also trap floatable debris, such as leaves and litter.

The second chamber also holds a permanent pool of water. An inverted pipe elbow leads to the third chamber which regulates water levels in the inlet. Runoff must pass through the bottom opening of the inverted pipe, and then travel upward several feet before it enters the third chamber. This design feature discourages clogging, and more importantly, traps oil and gas films floating on the surface in the second chamber. Oil and gas films remain in the second chamber until they are gradually adsorbed by sediment particles, and settle out.

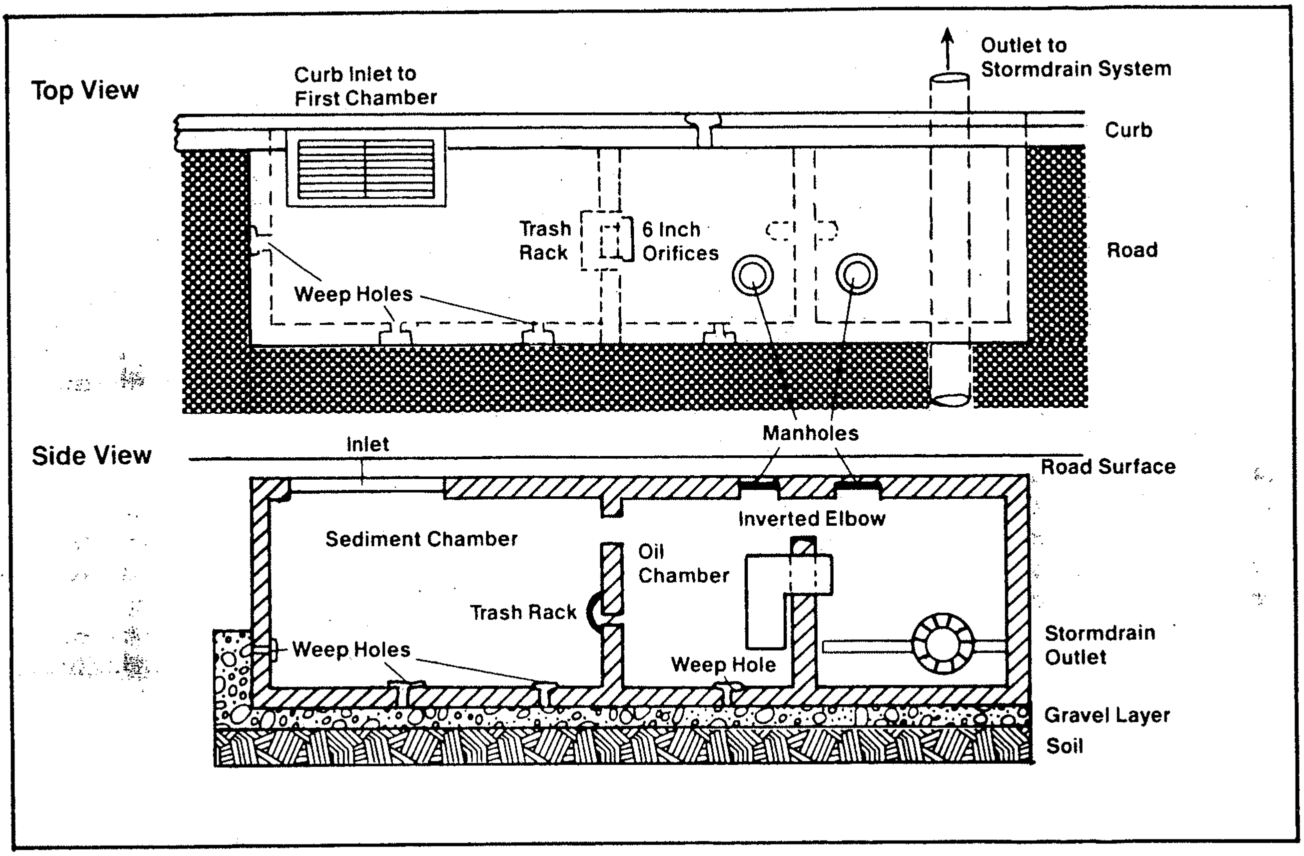
The third chamber contains a brick cradle that forms an opening to a storm drain outlet pipe. If the cradle is elevated from the floor of the chamber, a third permanent pool is created that may become an additional site for settling. Otherwise, the third chamber has little value in pollutant removal.

Water quality inlets are sized to provide 400 cubic feet of wet storage per contributing acre, and a pool at least four feet deep (MCDEP, 1984b). Additional dry storage must also be provided to pass the design storm. Access to each chamber for inspections and regular clean outs is provided by a separate manhole cover and step rungs.

Rockville Design

An alternative design for the three chamber inlet has been developed by the City of Rockville, Maryland (1984b). Runoff from a curb and gutter is directed into the first chamber of the inlet (Figure 8.2). However, unlike the Montgomery County design, the first and second chamber do not have a permanent pool. Instead, runoff drains through a series of screened six-inch weep holes situated on the floor of each chamber, then through a layer of stone aggregate, and eventually exfiltrates into the subsoil. Thus, the first and second chamber fill up only temporarily during storms, and then should drain completely. Pollutant removal is enhanced due to the exfiltration. However, since the weep holes are situated near the floor of the chamber, there are concerns about how long they can remain free of clogging from sediment deposits. In the event that the inlet does clog, it would then operate in the same manner as the conventional three-chamber inlet (i.e., a quasi-permanent pool would be formed in each chamber).

Figure 8.2: Schematic of a Water Quality Inlet, Rockville Percolating Inlet Design



Source: City of Rockville (1984b)

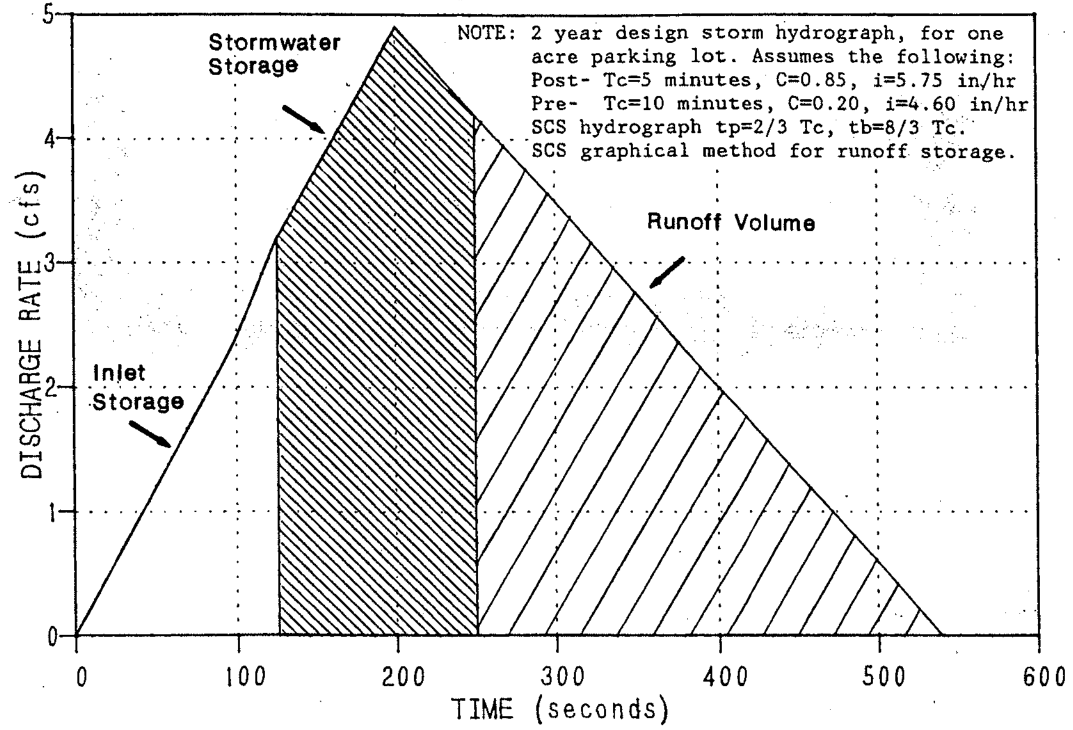
EFFECTIVENESS IN STORMWATER CONTROL

Because of their limited storage capacity, water quality inlets provide only marginal stormwater management benefits. In most instances, inlets are used to pretreat runoff before it is conveyed through the storm drain network, or into an infiltration trench or porous pavement facility. In very exceptional cases, water quality inlet design features can be incorporated into larger underground detention chambers to provide peak discharge control for a site.

Peak Discharge Control

Water quality inlets provide little or no control of post-development peak discharges, because of their limited storage capacity. Standard water quality inlets only have about 200 cubic feet of dry storage capacity per impervious acre. By way of contrast, the runoff volume needed to adequately control the two year design storm for the same impervious acre is over three times greater. As shown in Figure 8.3, standard sized water quality inlets provide only minimal storage of the design storm, and, consequently, should not greatly attenuate the peak discharge rate.

Figure 8.3: Size of Inlet Storage Compared to a Two Year Design Storm



NOTE: 2 year design storm hydrograph, for one acre parking lot. Assumes the following:
Post- Tc=5 minutes, C=0.85, i=5.75 in/hr
Pre- Tc=10 minutes, C=0.20, i=4.60 in/hr
SCS hydrograph tp=2/3 Tc, tb=8/3 Tc.
SCS graphical method for runoff storage.

Other Stormwater Benefits

For many of the same reasons noted above, the Montgomery County water quality inlet design will not provide significant groundwater recharge, low flow augmentation, runoff volume or streambank erosion control. The modest level of exfiltration that might be achieved by the modified Rockville inlet design may provide a marginal amount of groundwater recharge and volume control, if weep holes remain free from clogging.

POLLUTANT REMOVAL

The pollutant removal capability of water quality inlets has never been tested in the field, so their efficiency is largely a matter of speculation. Nonetheless, some general estimates as to their capabilities can be inferred from studies on similar structures (such as catchbasins and oil/water separators), as well as research on the settling behavior of sediment and hydrocarbons. However, field monitoring of this increasingly popular practice is badly needed.

A number of factors inherent in the three-chamber design serve to limit pollutant removal.

1. The limited amount of wet storage provided by the inlets. A standard sized three-chamber inlet has about 0.12 inches of runoff per acre in the permanent pool of the first and second chambers. By way of contrast, one-half inch of runoff storage per impervious acre is customarily recommended for pollutant removal of the first flush for other BMPs. In more concrete terms, the permanent pool is only about one-quarter of the size of the average storm in the Washington, D.C. area (approximately 0.45 inches of rainfall).

2. Since inlets serve such small areas, and have such a small capacity, runoff passes through them very quickly. The average detention time of runoff in an inlet during most storms will seldom exceed an hour, and in many cases, may be measured in minutes (see Figure 8.3).

3. Pollutants deposited within a chamber can only be permanently removed during clean-outs. Sediment deposited during smaller storms may be resuspended and scoured out during the next large storm (Pitt, 1985). Sediment tracer studies in catch basins (Pitt, 1985) indicate that only coarse-grained particles (such as grit, sand , some silt and debris) are likely to remain deposited for long periods. Pitt estimated that catch basins could removing about 10-25% of sediment and trace metals, and less than 10% of the nutrients in urban runoff if regularly cleaned. Field observations of accumulated sediments in three-chamber inlets (Galli, personal communication) suggest that inlets may also trap silt-sized particles, but it is not known whether they are prone to resuspension.

Despite these design limitations, there is reason to believe that inlets can help to remove coarse-grained sediments from urban runoff. For example, settling column studies (Grizzard et al., 1986) indicate that initial settlement of urban sediment is quite rapid, with about 20-40% dropping out within the first hour, depending on the initial sediment concentration (Figure 8.4).

Also, the design of water quality inlets should provide moderate removal of hydrocarbons. Since oil and gas are less dense than water, they initially float on the water surface. However, since oil and gas have a strong affinity for sediment, they rapidly adsorb to particles in the water column, and can then settle out.

DESIGN TIPS FOR ENHANCING POLLUTANT REMOVAL

Since the performance of water quality inlets has not been monitored, at this point, any discussion about enhancing pollutant removal is somewhat speculative. With this in mind, however, the following design tips are offered.

1. The wet pool volume in the first and second chambers of the inlet should be maximized. When feasible, use the third chamber as a permanent pool as well.

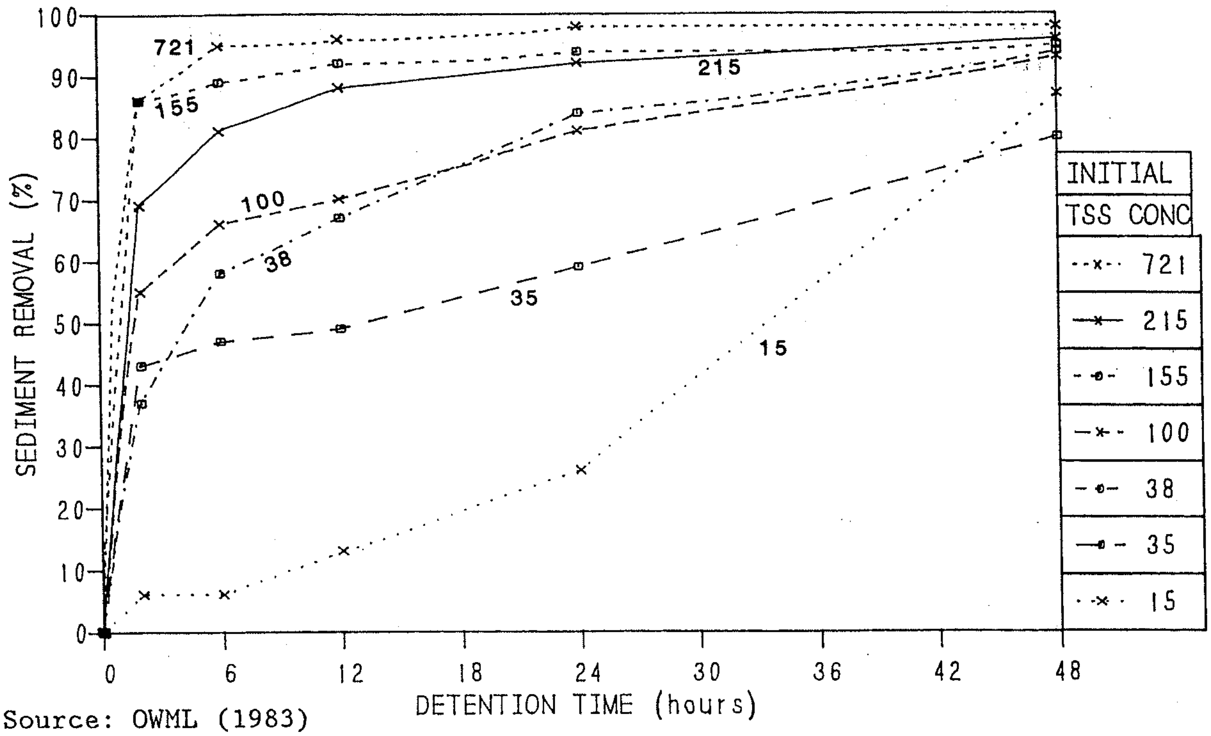
2. The orifice connecting the first chamber with the second should be protected from clogging by a trash rack welded to end plates fastened to the concrete sidewall. A typical design is presented in Figure 8.5.

3. The inverted elbow pipe connecting the second and third chambers should extend at least three feet down into the permanent pool to adequately separate out oil. A typical design is shown in Figure 8.5.

4. Resuspension of previously deposited sediment can be alleviated by installing baffle plates from the side walls to prevent the upward migration of sediments. In addition, the floor of each chamber should be slightly sloped away from the outlet to the next chamber, to help trap sediment.

5. Finally, as noted earlier, the only means of permanently removing pollutants from a water inlet is through regular clean-outs. As a general rule, the inlet should be cleaned out at least twice per year.

Figure 8.4: Sediment Settling Behavior for Urban Runoff



Source: OWML (1983)

PHYSICAL SUITABILITY AT THE SITE LEVEL

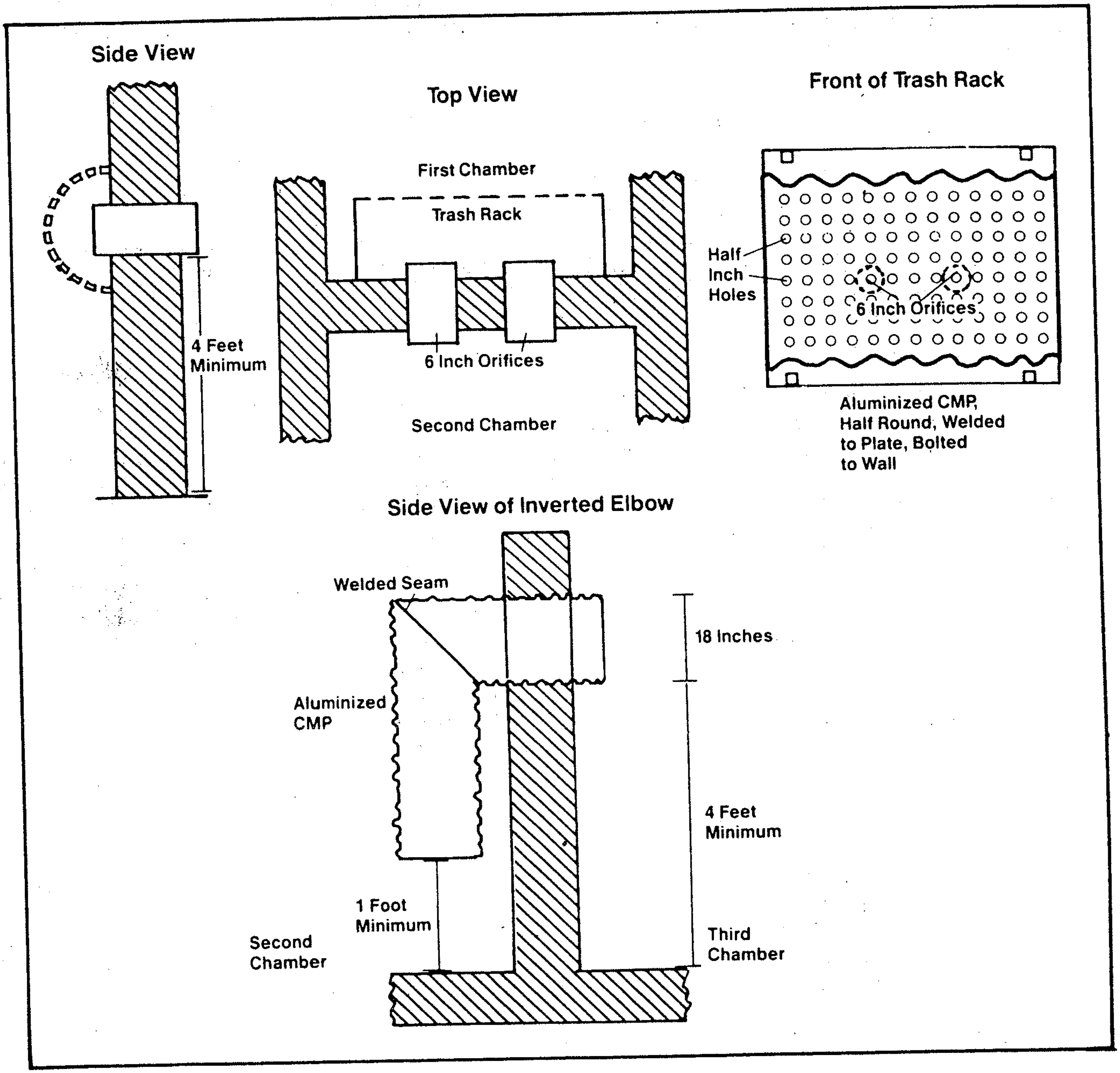
Water quality inlets are an adaptable practice for small sites, subject to the following constraints:

1. Either the inlet or the outlet from the water quality inlet must be connected to a storm drain network.

2. Individual inlets can normally only serve from a few tenths of an acre up to an acre each. As larger areas are served, the dimensions of the inlet, and consequently, installation costs, begin to climb rapidly.

3. If the Rockville percolating inlet design is being considered for use, the same factors used to evaluate the feasibility of infiltration trenches (soil permeability rates, depth to water table, proximity to foundations, etc.) should be checked prior to installation.

Figure 8.5: Detail on Trash Rack and Drawdown Pipe



Sources: WSSC (1986) and MCDEP (1984b)

WATER QUALITY INLET COSTS

The construction cost for a standard sized, three-chamber inlet design ranges between $5000-15,000 and averages about $7000-8000 (Galli, 1986b). The cost per inlet may drop sharply when pre-cast versions become available on the market.

The three-chamber design can also be incorporated into larger underground detention storage vaults that are sometimes used in development sites with little or no available land. As noted in Wiegand et al. (1986), underground detention chambers tend to be the most expensive BMP in the Washington region, in terms of cost per volume stored.

MAINTENANCE REQUIREMENTS

Water quality inlets must be cleaned out at least twice a year to accomplish any pollutant removal. The normal method used to clean out the Montgomery County design is topump out the contents of each chamber. The turbulence of the vacuum pump in the chamber produces a slurry of water and sediment that can then be transfered to a tank truck In some areas, the truck then disposes of the slurry into a sanitary sewer trunk line, where it travels to a treatment plant (MCDEP, 1984b).

An alternative disposal method is to carefully siphon out each chamber (without creating a slurry) and allow it to infiltrate over a nearby grass area. The remaining grit and sediments can then be removed manually, and trucked to a landfill for final disposal. Both disposal methods have significant drawbacks. Pumping of the inlets can be costly, and there is a risk of groundwater contamination associated with on-site siphoning.

In most cases, the public sector would be responsible for regular clean-outs and inlet inspections. Often this responsibility will fall to local public works departments, which have the equipment and manpower to perform the work, and which are already responsible for the maintenance of the storm drain network. Once inlets have been installed however, it would be important to keep centralized, up-to-date records on the location and status of each inlet in order to track maintenance and clean-out schedules.

RELEVANT DESIGN GUIDANCE

The following agencies can provide detailed plan specifications for water quality inlets:

Montgomery County Department of Environmental Protection, Stormwater Management Division, Rockville, Maryland.

Washington Suburban Sanitary Commission, Stormwater Management Planning Division, Laurel, Maryland.

City of Rockville, Public Works Department, Rockville, Maryland.

DESIGN SUMMARY: WATER QUALITY INLETS

- AREA SERVED:
  Inlets typically serve impervious areas of less than one acre.
- PERMANENT POOL:
  The volume of the permanent pool should be maximized. At least 400 cubic feet of wet storage per impervious acre is suggested as an initial sizing rule. The permanent pool in each chamber of the inlet should be at least four feet deep.
- CLEAN-OUT SCHEDULE:
  Accumulated sediment should be cleaned out from inlets at least twice per year. This can be done by vacuum pumping or siphoning of the permanent pool, and manually removing sediment deposits.
- DISPOSAL METHODS:
  Accumulated sediment deposits should be landfilled. Runoff in the inlet can be siphoned over to an adjacent grass filter strip, or transported to a sanitary sewer line and routed to a treatment plant.
- PREVENTING RESUSPENSION:
  Resuspension of deposited pollutants can be a problem in inlets. The use of vertical baffle plates on chamber floors may help alleviate this problem. Also, the floor of each chamber should slope slightly away from the outlet to the next chamber.
- INVERTED ELBOW:
  An inverted pipe with a 90 degree elbow should connect the second and third chambers of the inlet. The elbow can be formed by welding two cut sections of aluminized CMP, and the vertical portion should extend to one foot from the bottom of the inlet.
- USE WITH UNDERGROUND INFILTRATION:
  Inlets can be used to pretreat runoff before it enters an underground infiltration facility (e.g., porous pavement or an infiltration trench).
- CLOGGING:
  The two six-inch orifices that lead from the first to second chamber should be screened by a half round of aluminized CMP, in which half-inch holes have been drilled.
- ACCESS:
  To facilitate clean-outs, access to each chamber should be provided by means of a separate manhole and step rings.
- LAND USE:
  Water quality inlets are particularly appropriate for small redevelopment areas that generate high loads of sediment and hydrocarbons, such as service stations, convenience stores, and roads.

- **HYDRAULIC DESIGN:**

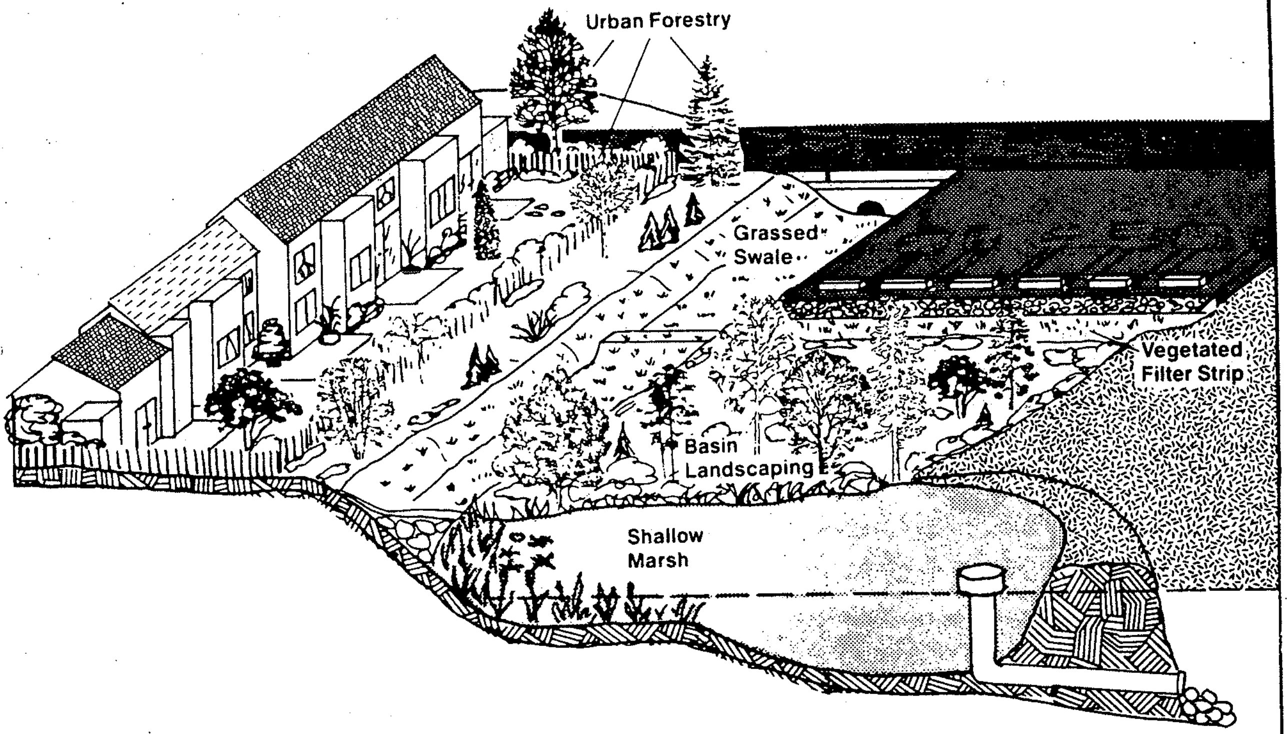
  The inlet should be designed to pass the two year design storm without hydraulic interference. This is normally done by placing a weir at least one foot above the water level in each chamber, and with at least a one foot gap to the top of the chamber.

CHAPTER 9: VEGETATIVE BMPs

This section reviews a diverse series of landscaping practices that can be applied to portions of the urban drainage system, including:

- Grassed Swales
- Filter Strips
- Urban Forestry
- Basin Landscaping
- Shallow Marsh Creation

Figure 9.1: Vegetative BMPs for a Site



All of these practices rely on various forms of vegetation to enhance the pollutant removal, habitat value, or appearance of a development site. While each practice by itself is not generally capable of entirely controlling the increased runoff and pollutant export from a site, they can improve the performance and amenity value of other BMPs, and should be considered as an integral part of every site plan. Typically, the costs for vegetative BMPs are very small in relation to those incurred when constructing ponds and basins. Also, vegetative BMPs can usually be applied during any stage of development, and in some instances, are attractive retrofit candidates.

Each of the vegetative practices is briefly reviewed in the following section; more detailed design guidance is provided by reference. The section concludes with a general landscaping guide for stormwater management areas.

GRASSED SWALES

General

Grassed swales are typically applied in single family residential developments and highway medians as an alternative to curb and gutter drainage systems (Figure 9.2). Swales have a limited capacity to accept runoff from large design storms, and often must lead into storm drain inlets to prevent large, concentrated flows from gullying/eroding the swale. If check dams are placed across the flow path, swales can provide some stormwater management for small design storms (Md WRA, 1984) by infiltration and flow attenuation. In most cases, however, swales must be used in combination with other BMPs downstream to meet stormwater management requirements.

Some modeling efforts and field studies indicate that swales can filter out particulate pollutants, under certain site conditions. However, swales are not generally capable of removing soluble pollutants, such as nutrients. In some cases, trace metals leached from swale culverts and nutrients leached from intensive lawn fertilization may actually increase the export of these pollutants. Grassed swales are usually less expensive than the curb and gutter alternative. Swale maintenance is performed by adjacent homeowners, and basically involves normal lawn activities such as mowing, watering, and chemical applications.

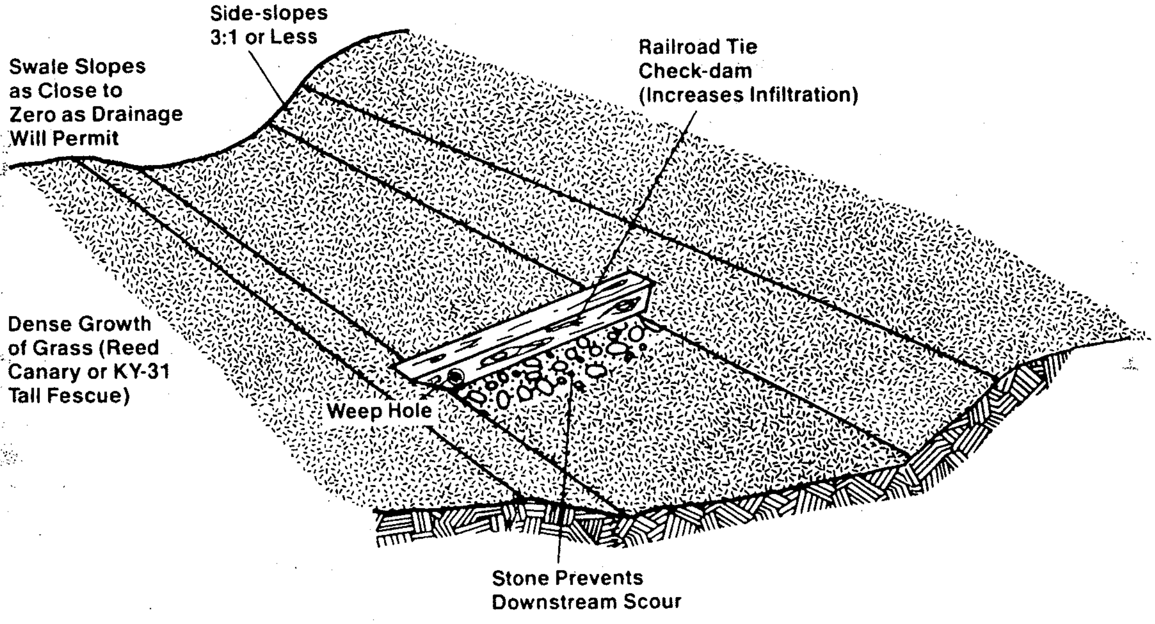
Stormwater Benefits

Swales act to control peak discharges in two ways. First, the grass reduces runoff velocity, depending on the length and slope of the swale. This, in turn, lengthens the watershed time of concentration (i.e., the time needed for runoff to reach the desired control point), and can, at least partially, attenuate the post-development peak discharge rate.

Second, a portion of the stormwater runoff volume passing through the swale infiltrates into the soil and does not appear at the downstream control point. However, as Wong and McCuen (1982) note, the volume of runoff that infiltrates into a swale is limited, seldom exceeding a few tenths of an inch, again depending on soils and slope. Runoff normally has a very short contact time in the swale (5-20 minutes) which does not give runoff much chance of infiltrating into the soil. Also, swale soils adjacent to roads are often heavily compacted to achieve desired slopes and load bearing strength, and as a result, have less infiltration capacity than undisturbed soils. In addition, the same rain that supplies runoff to a swale often has previously saturated the soils of the swale. Consequently, infiltration rates in a swale will almost always be near the minimum rates for the local soil type.

The hydrologic performance of swales can be improved if check dams are used to temporarily pond runoff. Appropriate design techniques are provided in Md WRA (1984).

Figure 9.2: Schematic of a Grassed Swale



Pollutant Removal

Pollutants are removed by the filtering action of the grass, deposition in low velocity areas, or by infiltration into the subsoil. Field monitoring has provided mixed results as to the extent of pollutant removal performed by swales. Kercher et al. (1983) and Yousef et al. (1985) reported moderate to high removal of particulate pollutants in low gradient, densely vegetated swales in Florida. In contrast, Oakland (1983) found low to moderate removal of particulate pollutants and negligible removal of soluble pollutants in a low-gradient swale, underlain by relatively impermeable soils in New Hampshire.

No statistically significant difference in runoff quality was observed at three residential, high gradient (2-5%) lawn swales monitored in the Washington, D.C. suburbs, when compared to curb and gutter controls (NVPDC, 1983). An alternative analysis of the same data by OWML (1983) provided some evidence that the swales may have slightly increased nutrient and trace metal export. Wigginton et al. (1986) examined trace metal enrichment patterns in soils of the same Washington area sites and found no evidence that metals had accumulated in the swales as a result of stormwater runoff. In fact, Wigginton and his colleagues discovered that metals were being leached from the roadway culverts that connected successive swales, perhaps as a result of the acidic nature of local rainfall. The mediocre performance of the Washington area swales was attributed to the rapid passage of runoff through the swales, soil compaction, high slopes, and short grass height.

Given the ambiguous nature of these studies, it is hard to propose specific estimates for swale pollutant removal efficiency. However, at least moderate removal of particulate pollutants can probably be expected during small storms, if a swale conforms to the following design considerations:

1. Swale slopes need to be graded as close to zero as drainage will permit. Side-slopes of the swale should be no greater than 3:1 (h:v).

2. A dense cover of a water tolerant, erosion resistant grass must be established. Reed canary grass is recommended for this purpose. Swale grasses should never be mowed close to the ground, as this impedes the filtering and hydraulic functions of the swale. Also, if a swale is adjacent to a roadway, sensitive species with a low salt tolerance (e.g., bluegrass) should be avoided.

3. Underlying soils need to have a high permeability (fc > 0.5 inches/hour). The swale should be tilled before the grass cover is established to restore infiltration capacity lost as a result of prior construction activities.

4. Check dams can be installed in swales to promote additional infiltration. The best method is to sink a railroad tie halfway into the swale, and place stone on the downstream side of the tie to prevent a scour hole from forming. Earthen check dams are not strongly recommended as they tend to erode on the downstream side (which may lead to the eventual wash out of the dam). It is also quite difficult to establish and maintain grass on earthen check dams. If a check dam is used, the designer should make sure that the maximum ponding time of runoff backed up behind the check dam is less than 24 hours (Md WRA, 1984).

Suitability

Many local jurisdictions prevent the use of grassed swales if peak discharges are expected to exceed 5 cfs, or if expected runoff velocities (computed using the Manning formula) are greater than 3 fps (Md WRA, 1984).

Swales are not likely to confer many stormwater or water quality benefits if constructed on slopes greater than 5%, or if groundwater extends to within two feet of the bottom of the swale.

Table 9.1: Comparative Costs For Vegetative Establishment

ESTABLISHMENT METHOD		AVERAGE COST PER ACRE			NOTES
		0-2 ac.	2-5 ac.	5+ ac.	
HYDROSEEDING:		$1,975	$1,750	$1,450	Permanent, guaranteed establishment, includes seedbed prep, mulch, anchoring, fertilizer, one post germination watering (Md WRA, 1985a)
CONVENTIONAL SEEDING:	(1)	$1,800	$1,650	$1,450	As above (Md WRA, 1985a)
	(2)	$8,475	--	--	For highly erodible areas that need a blanket or net during germination
SODDING:		$10,900	--	--	For Ky-31 Tall Fescue, Field Sod less costly; Bluegrass more costly
SWALES:					Excavation/shaping plus:
For a 15 ft wide, 3:1 sideslope swale	(1)	$4.50/linear foot:			Seeding/straw mulching
	(2)	$8.25/linear foot:			Seeding/net anchoring
	(3)	$7.75/linear foot:			Sodding/stapling
FORESTRY:	(1)	$100/acre (conifers)			Manual planting of seedlings, plus weed suppression
	(2)	$200/acre (deciduous)			
	(3)	$1000-5000/acre (variable)			Manual planting of nursery stock, depends on species, stock size
MARSH PLANTING:	(1)	$1000-3000/acre (estimated)			Rhizomes, plugs or small pots, and labor

Maintenance

Swale maintenance is largley aimed at keeping the grass cover dense and vigorous. This primarily involves periodic mowing, occasional spot reseeding, and weed control. Watering may also be necessary in times of drought, particularly in the first few months after establishment. Private homeowners are usually responsible for swale maintenance. Unfortunately, overzealous lawn maintenance on the part of homeowners can present some problems. For example, mowing the swale too close to the ground, and excessive application of fertilizers and pesticides, would all detrimentally affect the performance of the swale.

Costs

A strong advantage of grassed swales is that they are more economical than the curb and gutter drainage systems they replace (APWA, 1981). The comparative costs of establishing a permanent grass cover by various seeding methods have been documented by Md WRA (1985a), and these are detailed in Table 9.1

Environmental Attributes

Residential swales are essentially an extension of a front lawn, and have little wildlife or ecological value. Roadside and backyard swales, on the other hand, can be managed as a natural area. Over time, swales might be colonized by wetland plants and other shrubs that provide wildlife habitat and a more pleasing appearance. This process may be enhanced by intentional landscape plantings (see Landscaping Guide for appropriate species). However, a natural swale should never be confused with a neglected swale. A minimum level of maintenance (specifically, seasonal mowing) is still needed on any swale to prevent nuisance problems from developing, such mosquitos, ragweed, dumping, and erosion.

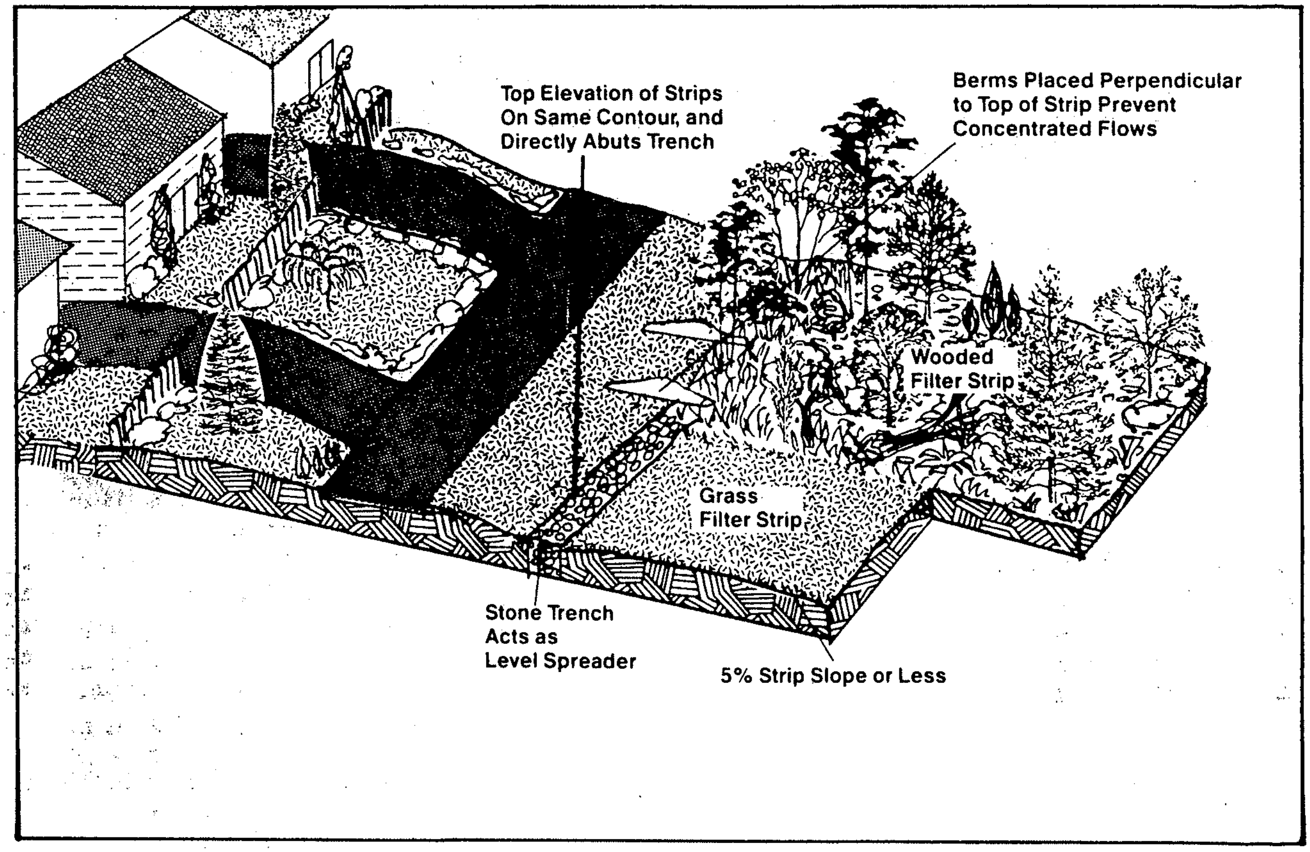
FILTER STRIPS

General

Filter strips are similar in many respects to grassed swales (Figure 9.3), except that they are designed to only accept overland sheet flow. Runoff from an adjacent impervious area must be evenly distributed across the filter strips. This is not an easy task, as runoff has a strong tendency to concentrate and form a channel. Once a channel is formed, the filter strip is effectively "short-circuited" and will not perform as designed. Such short-circuiting is a common problem. For example, over 60% of agricultural filter strips installed in Virginia were reported to have been short-circuited (Dilhalla et al., 1986).

To work properly, a filter strip must be 1) equipped with some sort of level spreading device, 2) densely vegetated with a mix of erosion resistant plant species that effectively bind the soil, 3) graded to a uniform, even, and relatively low slope, and 4) be at least as long as the contributing runoff area. Modeling studies indicate that filter strips built to these exacting specifications can remove a high percentage of particulate pollutants (Wong and McCuen, 1982). Much less is known about the capability of filter strips in removing soluble pollutants. Filter strips are relatively inexpensive to establish, and cost almost nothing if preserved before the site is developed. A creatively landscaped filter strip can become a valuable community amenity, providing wildlife habitat, screening, and stream protection. Grass filter strips are also extensively used to protect surface infiltration trenches from clogging by sediment.

Figure 9.3: Schematic of a Filter Strip



Stormwater Benefits

Filter strips do not provide enough storage or infiltration to effectively reduce peak discharges to predevelopment levels for design storms (Wong and McCuen, 1982). Typically, filter strips are viewed as one component in an integrated stormwater management system. Thus, the strips can lower runoff velocity (and, consequently, the watershed time of concentration), slightly reduce both runoff volume and watershed imperviousness, and contribute to groundwater recharge. At some sites, filter strips may help to reduce the size and cost of downstream control facilities. Filter strips are also of great value in preserving the riparian zone and stabilizing streambanks.

Pollutant Removal

Pollutant removal mechanisms in filter strips are similar to those discussed for grass swales. Results from some small test plots (Barfield et al., 1977) and several independent modeling studies (Wong and McCuen, 1982; Pitt, 1986, Overcash et al., 1981; Tollner et al., 1982) all suggest that filter strips are effective in removing particulate pollutants such as sediment, organic material and many trace metals. The rate of removal appears to be a function of the length, slope and soil permeability of the strip, the size of the contributing runoff area, and the runoff velocity.

From a design standpoint, the only variables that can be effectively manipulated are the length and slope of the strip. Md WRA (1984) suggest a minimum strip length of 20 feet; however, strips ranging in size from 100-300 feet are probably needed for adequate removal of the smaller sized sediment particles found in urban runoff (OWML, 1983). Wong and McCuen (1982) provide a useful and easily used model for sizing a filter strip, based on slope and channel parameters.

Removal of soluble pollutants in filter strips is accomplished when the pollutants infiltrate into the soil and are subsequently taken up by rooted vegetation. The efficiency of soluble pollutant removal in strips is probably not great since only a modest portion of the incoming runoff will be infiltrated (Wong and McCuen, 1982).

Forested filter strips appear to have greater pollutant removal capability than grassed filter strips, according to recent reports (Lowrance et al, 1984; Gilliam and Skaggs, 1983). A major reason cited for their efficiency is the greater uptake and long-term retention of nutrients in forest biomass (Brinson et al., 1981). However, because vegetative cover in forested strips is not as great as grass strips, they should probably be at least two times longer than a grass strip to achieve optimal removal.

As a final note, it should be remembered that the filter strip removal rates cited above are based on ideal conditions that may not always occur in the field (e.g., evenly distributed sheet flow, and uniform, dense and vigorous vegetation). If, for some reason, flow is allowed to concentrate before it reaches the filter strip, or as it crosses over it, removal rates will be significantly reduced. In severe cases, gully erosion within the filter strip may result in the strip becoming a net sediment source. The following design tips can help to prevent concentrated, erosive flows from forming in a strip:

1. The top edge of the filter strip should follow across the same elevational contour. If a section of the top edge of the strip dips below the contour, it is likely that runoff will eventually form a channel toward the low spot.

2. A shallow stone trench can be used as a level spreader at the top of the strip to distribute flow evenly. This also serves to protect the strip from man-made damage.

3. The top edge of the filter strip should directly abut the contributing impervious area. Otherwise, runoff may travel along the top of the filter strip, rather than through it. Dilhalla et al. (1986) suggest that berms be placed at 50-100 foot intervals perpendicular to the top edge of the strip to prevent runoff from bypassing the strip.

4. The appropriate length for filter strips is still the subject of some debate. As an absolute minimum, a grass strip should be at least 20 feet wide. Better performance can be achieved if the strip is 50-75 feet long, plus an additional four feet per each one percent of slope at the site (particularly if it is a forested strip).

5. Wooded filter strips are preferred to grassed strips. If an existing wooded belt cannot be preserved at the site, the grassed strip should be managed to gradually become wooded by intentional plantings.

6. If a filter strip has been used as a sediment control measure during the construction phase, it is advisable to regrade and reseed the top edge of the strip. Otherwise, the sediment trapped in the filter strip may affect the flow patterns across the strip, thereby reducing its effectiveness.

Suitability

Filter strips will not function as intended on slopes greater than 15%. These steeper slopes should still be vegetated but off-site runoff should be diverted around rather than through them. Filter strip performance is best on slopes with a grade of 5% or less. When the minimum length, twenty foot filter strips are used (for example, to protect a surface infiltration trench), slopes should be graded as close to zero as drainage permits.

To prevent concentrated flows from forming, it is advisable to have each filter strip serve a contributing area of five acres or less.

Maintenance

The maintenance required for a filter strip depends on whether or not natural vegetative succession is allowed to proceed. Under most conditions, the gradual transformation from grass to meadow to second growth forest will enhance rather than detract from the performance of longer filter strips. This process, which may be largely completed in a few decades, can be enhanced by intentional landscape plantings. Maintenance tasks and costs are both sharply reduced for these "natural" filter strips. However, corrective maintenance is still needed around the edge of the strip to prevent concentrated flows from forming.

Shorter filter strips must be managed as a lawn or short grass meadow. These strips should be mowed 2-3 times a year to suppress weeds and interrupt natural succession. Periodic spot repairs, watering and fertilization may be required to maintain a dense, vigorous growth of vegetation. Accumulated sediments deposited near the top of the strip will need to be manually removed over time to keep the original grade.

All filter strips should be inspected on an annual basis. Strips should be examined for damage by foot or vehicular traffic, encroachment, gully erosion, density of vegetation, and evidence of concentrated flows through or around the strip. Extra strip maintenance must be devoted in the first few months and years to make sure the strip becomes adequately established. This may involve extra watering, fertilization and reseeding.

Costs

The costs of establishing a filter strip are relatively low. Table 9.1 details the range in cost for permanent grass stabilization for several common seeding techniques (Md WRA, 1985a). Costs are negligible when an existing grass or meadow area is reserved at the site before development begins. Further savings can be realized if the filter strip is used as an on-site erosion control practice during the construction phase of development.

Environmental Attributes

Natural filter strips can provide excellent urban wildlife habitat, particularly for "edge" species of songbirds and mammals. Judicious planting of selected native trees, shrubs and grasses can be used to enhance the quality of food and cover (see Landscaping Guide at the end of the Chapter for a listing of appropriate species). Generally, much larger widths are needed for wildlife habitat purposes than for water quality purposes. For example, Stauffer and Best (1980) suggest that a 600 foot strip may be needed to support a full diversity of songbirds (although narrower strips will support a modest level of diversity). Even wider strips are needed to maintain species diversity for mammals and other terrestrial fauna (Brinson et al., 1981). The primary reason for wider strips is the well documented "island effect" on species diversity; i.e., the number and diversity of species present in a habitat is a function of the area of that habitat. From the human standpoint, a minimum 1000 foot wide strip has often been recommended for screening purposes and preservation of scenery. This roughly corresponds to the dimensions of existing stream valley parks in the Washington Area.

Relevant Design Guidance

Maryland Soil Conservation Service, 1983. Standards and Specifications For Soil Erosion and Sediment Control. Maryland Water Resources Administration, Annapolis, Maryland.

Maryland Department of Natural Resources, 1984. Standards and Specifications For Stormwater Management Infiltration Practices. Water Resources Administration, Annapolis, Maryland.

Virginia Department of Soil and Water Conservation, 1972. Best Management Practices Handbook: Agriculture. Planning Bulletin No. 316. Virginia State Water Control Board, Richmond, Virginia.

URBAN FORESTRY

General

Urban forestry involves either preserving trees during construction, planting them after the site has been cleared, or homeowner landscaping after the site has been fully developed. With careful landscape design, as much as 50% of a residential lot can be converted into an attractive natural setting of trees, shrubs and ground covers. The amount of runoff generated from these landscaped areas is often 30-50% less than that produced from turf or lawns. Pollutant removal through urban forestry, on the other hand, is limited (with the notable exception of forested buffer strips). The cost and maintenance requirements for most urban forestry practices are quite low, yet the environmental amenity value is often very high.

Stormwater Benefits

Trees, shrubs and ground covers provide many stormwater management benefits. When mature, these plants form a canopy that intercepts much rainfall before it reaches the ground. Rainfall that does reach the ground is more likely to be infiltrated in the spongy layer of organic matter that accumulates underneath the plants. Consequently, both the volume and peak rate of stormwater runoff are reduced. Pitt et al. (1986) suggest that forested areas may produce 30-50% less runoff than grassed lawns.

Pollutant Removal

Urban forestry can help remove pollutants in a number of ways: by plant uptake and storage, by reducing the volume of storm runoff (and associated pollutants) delivered from the site, and by preventing soil erosion. However, the actual impact of urban forestry in reducing pollutant export from the site is probably limited for two reasons. First, as noted in Chapter 1, the bulk of the pollutant load from urban areas is generated from impervious areas, and not the pervious lawn areas that are the focus of urban forestry. Second, if a tree does extend over an impervious surface, it may become a potential source of pollutants, either by deposition of nutrient rich pollen, leaching of soluble nutrients from the canopy leaves, or during autumn leaf fall.

Suitability

Reforestation or forest preservation measures can and should be applied to all development areas, both existing and planned. The trick involves selecting the proper species and mix of trees and shrubs that are best suited to the unique growing conditions at the site. Factors such as the soil pH, texture, moisture and fertility need to be taken into account, as well as the current and expected level of exposure and sunlight. Trees that grow well in wet and moist soils are indicated in the Stormwater Landscaping Guide (Table 9.2). State foresters, cooperative extension agents, and nurserymen can help select the species which are best adapted for the site, and can suggest appropriate soil testing procedures.

Reforestation measures should not be considered for areas expected to receive a great deal of foot traffic, such as playgrounds or walkways. Only grass turf is resilient enough to withstand frequent traffic; use of natural ground covers in these areas will quickly result in barren patches that may erode.

Maintenance

Urban forestry measures require very little concerted maintenance, except perhaps during the first few years after establishment, where weed suppression, rodent protection, watering, or staking may be needed to increase survival rates. Thereafter, maintenance requirements are minimal.

Costs

The costs of urban forestry measures are variable. Costs are essentially nonexistent if trees are preserved during the land clearing phase, although some nominal costs may be incurred in selecting the trees to be kept, and more importantly, keeping heavy equipment from damaging their trunks and roots during construction. The most economical (and least rapid) means of reforestation is to plant seedlings. These can be readily obtained from state tree nurseries, and range in cost from $20 to $120 per thousand, depending on the species selected. As a rule of thumb, to attain maximum crown closure, about 700 deciduous seedlings are needed per acre (8'x 8'spacing); whereas about 1200 seedlings/acre are needed for conifer species (6'x 6' spacing)(Meckley et al., 1986). Manual planting costs, including weed suppression, generally run about $100-200 per acre planted.

A more common and more rapid means of reforestation in the region involves planting saplings or nursery stock. This approach is more costly, due to the greater expense of nursery stock and the labor required to prepare and plant each site. Although landscaping costs will vary significantly depending on size and species selected, total costs on the order of $1000-5000 per acre are not uncommon.

Urban forestry confers several environmental amenities that can help to offset the initial planting costs. These include the well documented effect of landscaping on property values, reduced heating and cooling bills for the home, and reduced effort and expense needed to maintain lawns (Pitt et al., 1986).

Environmental Attributes

Urban forestry improves the quality of the natural environment in a number of ways, most notably, in the provision of food, cover and nesting sites for wildlife. Shade trees planted next to low flow channels or streams can help keep water temperatures cool and thus protect aquatic habitat. Trees planted in the riparian zone can also stabilize streambanks to minimize erosion.

Urban forestry has beneficial impacts on the human environment as well. Trees can help to absorb noise, provide shade, screen scenery, break the wind, moderate local air temperatures and improve the landscaping value of a site.

Relevant Design Guidance

Additional guidance on urban forestry can be obtained through the local offices of the State Forester or the Cooperative Extension Agent.

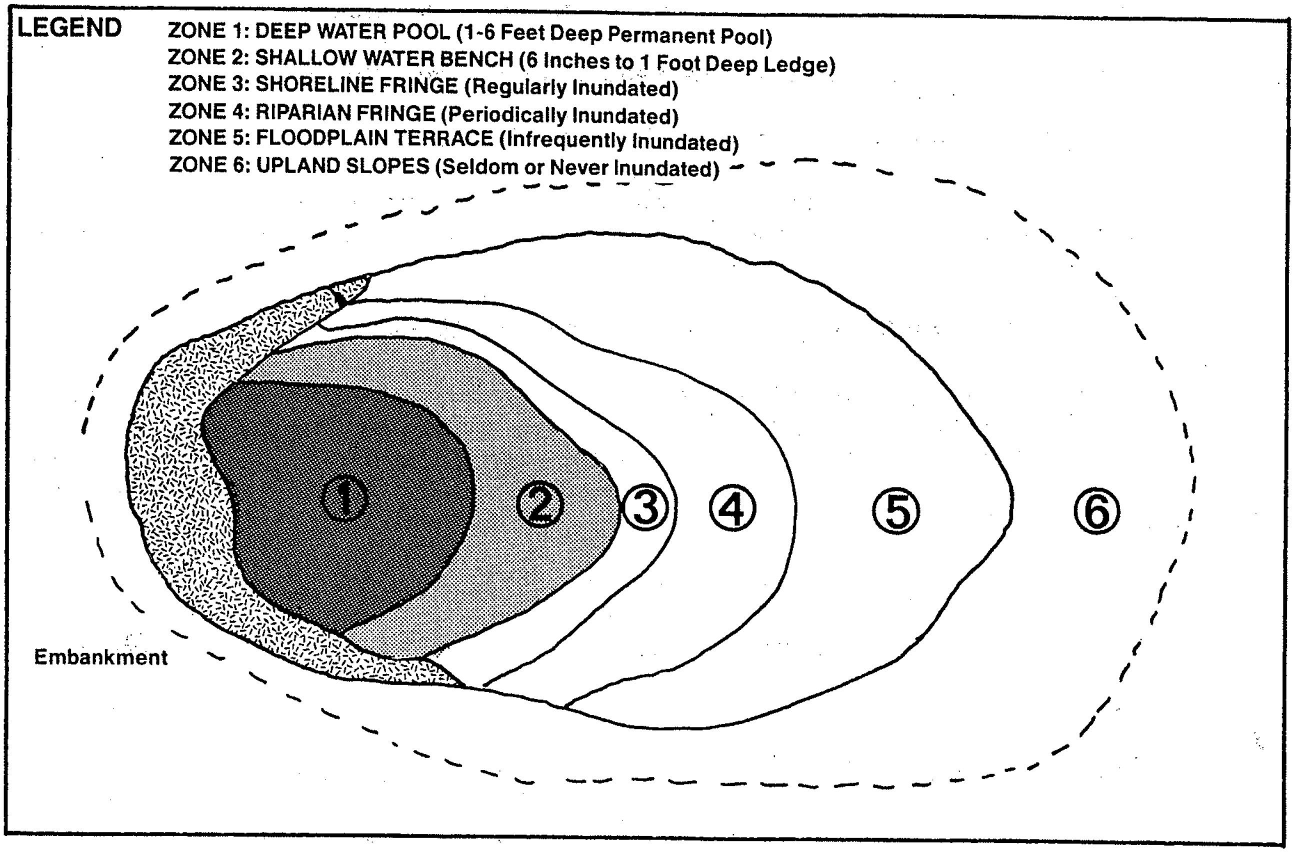
BASIN LANDSCAPING

General

Landscaping is a critical element in the design of stormwater basins, whether they are dry ponds, extended detention ponds, infiltration basins or wet ponds. Every basin design should be accompanied by a landscaping plan, as the form and species of plants used to stabilize a basin has a profound influence on its pollutant removal performance, appearance, habitat value and maintenance requirements well into the future.

Due to differences in the frequency and depth of inundation during storms, a wide gradient in soil moisture conditions is formed around a pond. Each plant species has a rather narrow tolerance with respect to soil moisture, and consequently,species tend to cluster in areas that best meet their special growth requirements. As shown in Figure 9.4, as many as six distinct vegetative zones may be present in a single pond. Plant species selected for each zone can fulfill specific functional design objectives. These are described below.

Figure 9.4: Landscaping Zones Around a Stormwater Basin



Zone 1: Deep Water Areas (wet ponds only)

The permanent pool can be planted with submerged aquatic plants to enhance pollutant uptake and provide food for waterfowl. However, artificial establishment of submerged plants in wet ponds has not been routinely practiced in the region because submerged plants are not easy to obtain, may be difficult to establish, and could potentially pose a clogging problem. Some species that might be used are duckweed, wild celery, sago pondweed and redhead grass (Barnard, personal communication). As the names suggest, these species have a high food value for waterfowl.

Zone 2: Shallow Water Areas (wet ponds and shallow marshes)

The shallow aquatic bench around a wet pond, with an average depth of a foot or less, favors the growth of emergent aquatic vegetation. This fringe of vegetation can perform a wide variety of design functions:

1. Enhance nutrient uptake within the wet pond.

2. Reduce water velocities and increase local sediment deposition rates.

3. Stabilize bottom sediments and reduce wind induced sediment resuspension.

4. Provide valuable food and cover for wildlife, particularly waterfowl and shorebirds.

5. Form a diverse habitat for predatory insects, that act as a natural check on unwelcome mosquitos.

6. Conceal trash and floating objects blown toward shore.

7. Absorb wave impacts, and thus reduce shoreline erosion.

8. Disguise unaesthetic changes in water level.

The use of emergent vegetation should be seriously considered in wet pond designs. Additional guidance can be found in the section on shallow marsh establishment.

Zone 3: Pond Shoreline (wet pond and shallow marsh)

Plants growing along the shoreline of a pond can be expected to be routinely inundated during most storms and on windy days. However, the plants must also be capable of withstanding periodic drying during extended pond drawdowns in the summer months. Plants selected for this zone can accomplish the following design objectives:

1. Stabilize the shoreline from erosion.

2. Conceal changes in water levels.

3. Prevent or limit access to the pond.

4. Provide cover, nesting and loafing areas for waterfowl.

5. Provide food for waterfowl, songbirds, and terrestrial wildlife.

6. Shade the surface of the pond to mitigate pond warming.

Zone 4: Riparian Fringe Area (lower stage of extended detention ponds, infiltration basins and dry ponds)

Plants in this zone must be able to tolerate wet soil conditions most of the time and can expect to be inundated for brief periods during most storms. A variety of trees, shrubs and ground covers can be utilized to achieve the following design objectives:

1. Stabilize the floor of the basin to prevent erosion.

2. Bind up newly deposited sediments to prevent their resuspension.

3. Reduce local water velocities to induce better settling characteristics within the pond.

4. Conceal and trap trash, debris and other floating objects washed into the pond.

5. Create habitat and food for wildlife.

6. Keep the soils of the basin floor drier by increased plant transpiration.

7. Maintain soil infiltration capacity in infiltration basins through root penetration.

Zone 5: Floodplain Terrace (upper stage of all basins and along stream channels).

This zone is subject to periodic inundation, but only during infrequent large storms. Plants that grow in this zone can tolerate infrequent inundation, but prefer only moist or slightly wet soil conditions most of the year. This zone roughly corresponds to the area of the pond between the one and five year water surface elevation. Plant species native to floodplains usually grow very well in this zone. Since the zone encompasses much of the embankment and side-slopes of a basin, plant selection and siting should be oriented toward:

1. Ground covers that can prevent erosion on steep slopes, and need not be mowed often.

2. Strategic placement of trees and shrubs to break up the engineered contours of the basin.

3. Plants that do not conflict with the intended use of the open space (e.g., visual appeal, recreation, wildlife habitat, access regulation, and foot traffic).

4. Species that tolerate exposure, compacted soils and have minimal maintenance requirements.

Zone 6: Upland Areas (buffer areas for all basins).

The unifying feature of plants that grow in this zone is that they are seldom, if ever, inundated. However, the environmental conditions and management strategy for the pond buffer will differ at each development site. Appropriate plant species selected for the buffer will depend on local soil conditions, exposure levels and the intended use of the open space.

The stormwater management landscaping guide presented in the last section provides a list of suitable plant species keyed to each of the major vegetative zones around a basin.

Basin Landscaping Tips

Stormwater management basins are often a harsh environment for establishing vegetation. The following landscaping procedures should enhance survival rates.

1. Trees with rootballs greater than 30 inches should never be planted on the pond embankments (Md SCS, 1976). The root systems of large trees can threaten the structural integrity of the embankment.

2. Embankments and basin side-slopes are often compacted during construction to ensure structural stability. The density of compacted soils normally prevents extensive root penetration. Therefore, larger holes must be dug and backfilled with uncompacted soil to accommodate the root systems of trees and shrubs (Wittans and Wiess, 1985).

3. Most newly constructed stormwater management basins will be fully exposed for a number of years. Thus, plant species that require shade, are susceptible to winterkill, or are prone to wind damage should be avoided.

4. The use of regionally native plant species is recommended. These plants are better adapted to local climate and soil conditions, and are thus more likely to survive. Native plants also tend to need less maintenance than exotic or ornamental plants. Some sources of native plants seeds and nursery stock are provided in the next section.

5. The first few years after planting are critical. Extra maintenance (watering, support, fertilizing, mulching, weed suppression) is required to nurture the plants through this difficult phase.

Sources of Plant Materials

Soil Conservation Society of America. 1982. Sources of Native Seeds and Plants. 7515 Northeast Ankeny Road, Ankeny, Iowa. (272 retail and wholesale outlets listed by State and plant specialty; $3.00).

Environmental Concern, Inc. P.O. Box P, 210 West Chew Avenue, St. Michaels, Maryland 21663. (Catalogs available upon request, specializes in aquatic plants, also carries native trees and shrubs).

National Arboretum. 3501 New York Avenue, N.E. Washington, D.C. 20002. (List of 30 wholesale/retail outlets that propagate native plants).

Lilypons Water Garden. 6800 Lilypons Road, Lilypons, MD 21717. (Catalog of primarily aquatic plants).

Restoration and Management Notes. 114 N. Murray Street, Madison, WI 53715 (Quarterly journal covers developments in wetland and meadow management, numerous native plant nurseries advertise in each issue).

Wildlife Nurseries, P.O. Box 2724, Oshkosh, WI. 54903. (Source for submerged aquatic plant seeds).

SHALLOW MARSH CREATION

General

Wetland or shallow marsh creation is basically a form of basin landscaping. It is given special treatment here due to its important role in pollutant removal and the need for careful design. Wetlands can be established around the perimeter of a wet pond, the lower stage of an extended detention pond, or in a sediment forebay. The many benefits of wetlands have been summarized in earlier sections. Athanas (1986), Lakatos and McNemar (1986), and Md SCS (1986) have compiled design guidance for successful wetland establishment, which have been abstracted below:

Plant Propagation Methods

While emergent plants may eventually colonize a basin from upstream wetland areas or by waterfowl, the most reliable means of establishing a marsh is to transplant live plants or dormant rhizomes from nursery stock. Transplantation from existing wetlands is not as effective and can result in serious impacts to the source marsh. Seeding, while cheaper, is also not a reliable means of propagating wetland plants due to their tricky germination requirements. A number of commercial outlets carry wetland plant stock, some reliable sources are provided on the previous page.

Water Depth Requirements

Most wetland species have very specific water depth requirements. In general, most species thrive in shallow water conditions less than a foot deep (six inches is probably optimal). To achieve these depths over a wide area, sites will often have to be regraded. Also, potential sites for wetland establishment must have sufficient baseflow to maintain a relatively constant water level. Otherwise, severe seasonal drawdowns will occur at the site that will adversely affect the growth and colonization rate of wetland plants.

Aquatic Bench/Sediment Forebay

The perimeter of a wet pond should be graded to form a 10 to 20 foot wide shallow bench for aquatic emergents. It is recommended that the bench extend around at least half of the pond's perimeter. Also, the pond can be excavated to provide a shallow area for marsh establishment (and sediment deposition) near the inflow channel.

Marsh Surface Area

Optimal nutrient removal performance is achieved by shallow marshes when the surface area of the marsh is maximized. As a general guideline, the surface area of the marsh should constitute about 2 to 3% of the total area of the contributing watershed. Alternately, the nutrient mass loading method described in Example 3.2 of Chapter 3 can be used to determine the acceptable area for the marsh.

Planting Strategies

At least two primary wetland species, which are hardy and rapid colonizers, should be planted over about 30% of the total shallow water area. Each primary species should be planted in three or four monospecific stands, with individual plants about 2 to 3 feet apart. Up to three secondary wetland species, that are not as aggressive in colonizing a pond, should be randomly distributed in clumps around the perimeter of the marsh. Primary and secondary wetland species are listed in the landscaping guide, and a schematic of the planting strategy is shown in Figure 9.5. This planting strategy is designed to take advantage of natural propagation to fill out the rest of the marsh. Also, since a diverse number of wetland species are utilized, the strategy minimizes the risk that the marsh will not become successfully established.

Open Water Areas

If a basin is exclusively designed to act as a shallow marsh, at least 25% of the total surface area of the inundated area should be reserved for open water areas that are two or more feet deep. This combination of marsh and open water provides ideal habitat for waterfowl and marshbirds (Md SCS, 1986) and is more visually appealing.

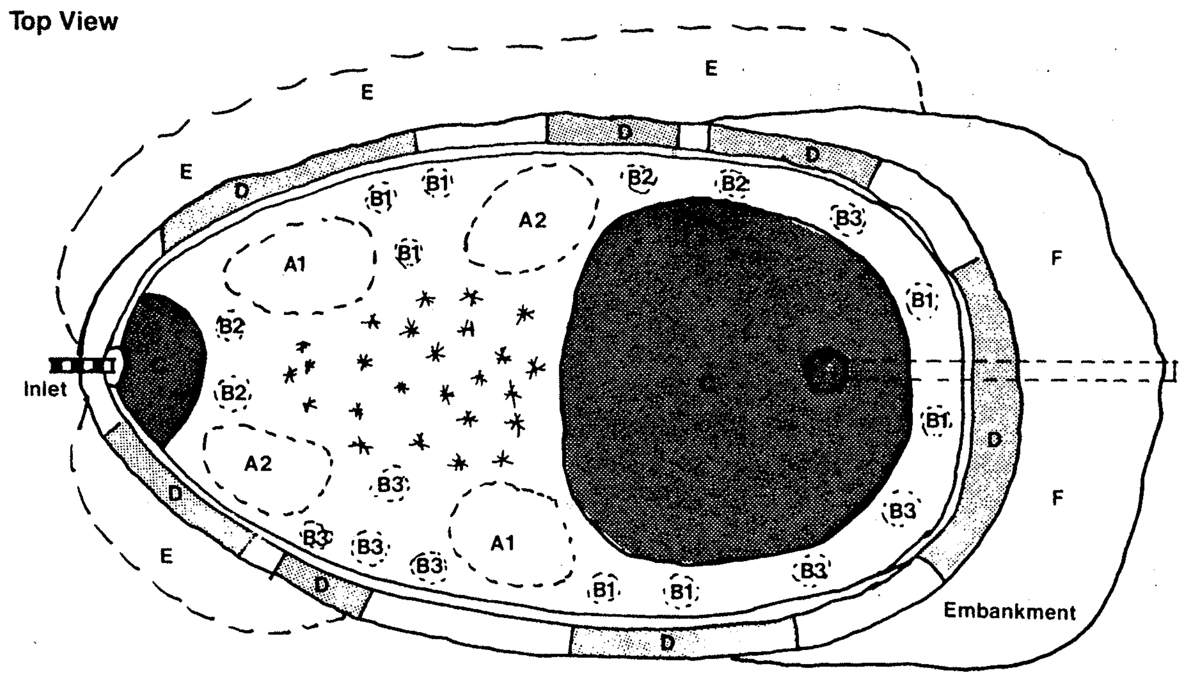
Design Guidance

Athanas, C., 1986. Wetland Basins for Stormwater Treatment. Horn Point Environmental Labs, University of Maryland. prepared for Md DNR, Annapolis, Maryland.

Maryland DNR, 1987. Design Guidelines For Shallow Marshes. Maryland Water Resources Administration, Annapolis, Maryland.

Soil Conservation Service. 1986. Technical Guide For Wetland Management. Maryland Field Office.

Figure 9.5: Examples of Shallow Marsh Planting Strategies



LEGEND

A1: Primary Wetland Species fi1

A2: Primary Wetland Species fi2

B1: Secondary Wetland Species fi1

B2: Secondary Wetland Species fi2

B3: Secondary Wetland Species fi3

C: Deep Open Water Areas (No Vegetation)

D: Shoreline Fringe of Switchgrass, 50% of Safety Bench Perimeter

E: Wildlife Trees and Shrubs In Buffer

F: Maintain Grass Cover On Embankment (No Trees)

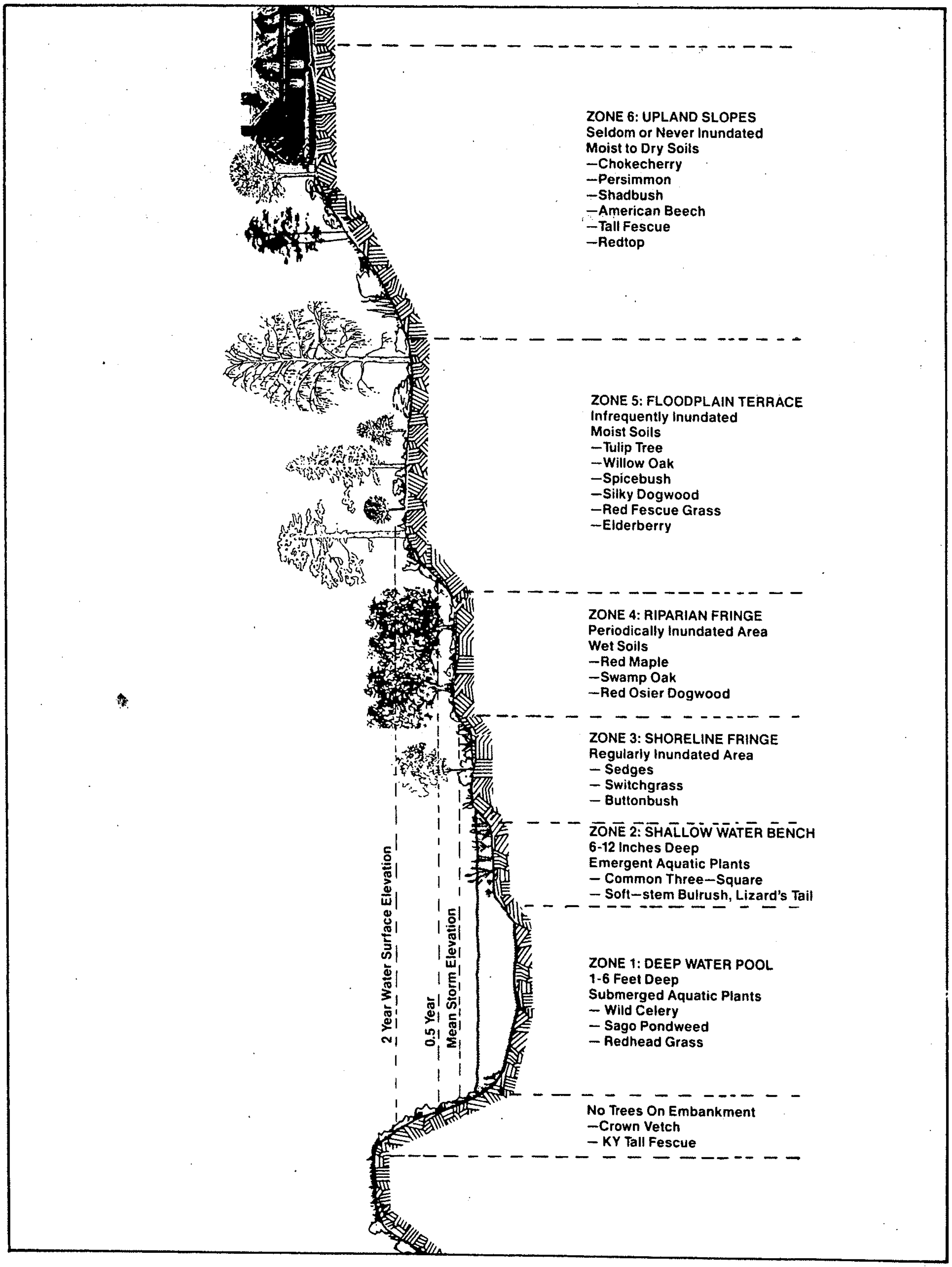
Adapted From Athanas, (1986).

A GUIDE TO URBAN LANDSCAPING

As noted earlier, differences in soil moisture and the frequency of inundation create several distinct vegetative zones around a pond (Figure 9.6). Species of trees, shrubs ground covers and aquatic plants that are suited to each of these zones are provided in Table 9.2. The plant list, which has been compiled from a number of sources, indicates the common and Latin names of each plant, as well as information on its form, height, commercial availability, capability to withstand inundation, wildlife value, and special growth requirements. The list only includes a few plant species that grow best on drier or better drained soils; these species are treated in detail in more conventional landscaping guides.

Finally, Table 9.2 is intended to only provide general guidance on plant selection. The advice of qualified landscape architects, nurserymen, or extension agents should be consulted to select the best plants for specific applications.

Figure 9.6: Key to Landscaping Zones in Stormwater Areas



Adapted from Wittans and Weiss, 1985

Table 9.2: Landscaping Guide for Stormwater Management Areas

Plant Name Common (Latin)	Zone	Form	Availability	Tolerance for Periodic Inundation	Native Plant	Wildlife Value	Special Requirements	Notes
TREES AND SHRUBS:								
Alder (Alnus glutinosa)	5,6	Decid. Tree up to 50'	No	Yes	No			Rapid growing, stabilizes stream banks
American Beech (Fagus grandifolia)	5,6	Decid. tree	Yes	No	Yes	High, mammals, birds	Prefers shade, rich, well-drained loam soils	
American Holly (Ilex opaca)	5,6	Decid. Shrub to 40-50'	Yes	No	Yes	Yes, songbirds, food, nesting	Prefers shade, tolerates periodic drought	Ornamental
Autumn Olive (Elaeagnus umbellata)	6	Decid. Shrub 10-20'	Yes	No	No	High, food and cover rapid growing	Full sun, well-drained soil	Useful in hedgerows/ aggressive
Blackgum (Nyssa sylvatica)	4,5,6	Decid. Tree 30-60'	Yes	Yes	Yes	High, songbirds	Prefers sun	Can be used as ornamental
Black Willow (Salix nigra)	3,4,5	Decid. Tree 30-50'	Yes	Yes	Yes	Low, some browsing and cavity nesters	Full sun	Rapid growth, stabilizes streambanks, weak
Buttonbush (Cephalanthus occidentalis)	2,3,4,5	Decid. Shrub 6-9'	Yes	Yes	Yes	High, ducks and shorebirds	Full sun to partial shade	
Chokecherry (Prunus virginiana)	5,6	Decid. Shrub 6-20'	Yes	No	Yes	High, birds and mammals	Well-drained to moist soils	
Elderberry (Sambucus canadensis)	4,5,6	Decid. Shrub 3-12'	Yes	Yes	Yes	Extremely high for food and cover	Full sun to partial shade	
Fringe Tree (Chionanthus virginicus)	3,4,5	Decid. Tree 10-20'	Yes	Probably	Yes	Unknown	" "	Ornamental
Green Ash, Red Ash (Fraxinus pensylvanica)	4,5	Decid. Tree 30-80'	Yes	Yes	Yes	Moderate, songbirds	Full sun	Rapid growing streambank stabilizer
Honey Locust (Gleditisia triacanthos)	4,5,6	Decid. Tree 70-80'	Yes	No	No	Low	Full sun	
High Bush Cranberry (Viburnum trilobum)	4,5,6	Decid. Shrub 10'	Difficult	Yes	No	Moderate, emergency winter food	Full sun	Acidic soils only
Larch/Tamarack (Larix laricina)	3,4	Conifer. Tree 20-40'	Only as L. decidua	Yes	Yes	Low	Full sun, acidic boggy soils	Rapid initial growth

(Table 9.2 cont.)

Plant Name Common (Latin)	Zone	Form	Avail-ability	Tolerance for Periodic Inundation	Native Plant	Wildlife Value	Special Requirements	Notes
Mountain Laurel (Kalmia latifolia)	6	Conifer. Shrub 5-10'	Yes	No	Yes	Low	Partial shade, acidic soils	Ornamental
Persimmon (Diospyros virginiana)	4,5,6	Decid. Tree 30'	Yes	No	Yes	Extremely high, birds, mammals	Well-drained soils	Not shade tolerant
Red Chokeberry (Pyrus arbutifolia)	3,4,5	Decid. Shrub 2-8'	Yes	Yes	Yes	Moderate, song-birds	Partial sun	
Red Maple (Acer rubrum)	4,5,6	Decid. Tree 40-70'	Yes	Yes	Yes	High, seeds and browse		Rapid growth
Red Osier Dogwood (Cornus Stolonifera)	3,4,5	Decid. Shrub 4-8'	Yes	Yes	No	Moderate, song-birds	Shade tolerant	Bank stabilizer
Rhodendron spp.	4,5,6	Conifer. shrub 5-12'	Yes	Only as R. viscosum	Yes	Low	Acid soil, shade	Ornamental
River Birch (Betula nigra)	3,4	Decid. tree 20-40', to 90'	Yes	Yes	Yes	Low, but good for cavity nesters		Bank erosion control
Shadbush, Common Serviceberry (Amelanchier arborea)	5,6	Decid. shrub 15-20'	Yes	Yes	Yes	High, birds, mammals	Prefers shade	
Silky Dogwood (Cornus amomum)	5,6	Decid. shrub 4-10'	Yes	Yes	Yes	Moderate, song-birds	Shade, drought tolerant	
Silver Maple (Acer saccharinum)	4,5,6	Decid. tree 60-80'	Yes	Yes	Yes	Moderate, song-birds		Ornamental, weak wood
Southern Arrowwood (Viburnum dentatum)	4,5	Decid. Shrub to 10'	Yes	No	Yes	Yes, songbirds	Partial sun	
Spice bush (Lindera benzoin)	5,6	Decid. shrub 12-25'	Yes	No	Yes	Moderate, song-birds	Shade, rich soils	
Swamp Magnolia or Sweetbay (Magnolia virginiana)	3,4	Conifer. Tree 20'	Yes	Yes	Yes	Low	Shade	Ornamental
Swamp Oak (Quercus bicolor)	4,5	Decid. tree 60'	No	Yes	Yes	High, mast		
Sweetgum (Liquidambar styraciflua)	4,5,6	Decid. tree 50-70'	Yes	Yes	Yes	Moderate, song-birds		Tolerates acid or clay soils
Sycamore (Platanus occidentalis)	4,5,6	Decid. tree 80'	Yes	Yes	Yes	Low		Rapid growth

(Table 9.2 cont.)

Plant Name Common (Latin)	Zone	Form	Avail-ability	Tolerance for Periodic Inundation	Native Plant	Wildlife Value	Special Requirements	Notes
Tulip Tree (Liriodendron tulipifera)	5	Decid. tree 70'	Yes	No	Yes	Moderate, birds, mammals		Rapid growth
Tupelo (Nyssa sylvatica vari biflora)	3,4,5	Decid. tree 35'	Yes	Yes	Yes	High, seeds, cavity nesters		Ornamental
Weeping Birch (Betula pendulata)	4,5,6	Decid. tree 25'	Yes	Yes	No	Moderate		Ornamental
Willow Oak/Pin Oak (Quercus phellos/ palustris)	4,5,6	Decid. tree 50-90'	Yes	No - phellos Yes - palustrus	Yes	High, mast		
Winterberry (Ilex laevigata)	4,5	Decid. Shrub 8-10'	Yes	No	Yes	Moderate, song-birds		
Witch Hazel (Hamamelis virginiana)	4,5	Decid. shrub 10'	Yes	No	Yes	Low	Shade	Ornamental
WETLAND PLANTS:								
Arrow Arum/Duck Corn (Peltandra virginica)	2	Emergent	Yes	to @ 1 ft. depth	Yes	Low, except for wood ducks		Slow colonizer
Arrowhead/Duck Potato (Saggitaria latifolia)	2	Emergent	Yes	to @ 1-1.5 ft. depth	Yes	Moderate, ducks		Aggressive colonizer
Buttonbush (Cephalanthus occidentalis)	2,3	Emergent	Yes	to @ 2 ft. depth	Yes	High, ducks and and shorebirds	Full sun	
Broomsedge (Andropogon virginianus)	2,3	Perimeter	Yes	to @ 3 in. depth	Yes	Moderate, song-birds, browsers	tolerates fluctuating water levels	
Cattail (Typha spp.)	2,3	Emergent	Yes	to 1 ft. depth	Yes	Low, except as cover		Volunteer, aggressive colonizer
Coontail (Ceratophyllum demersum)	1	Submergent	No	1-6 ft. deep	Yes	Low		
Common Three-Square (Scirpus americanus)	2,	Emergent	Yes	to 6 in. deep	Yes	High, waterfowl, song-birds	Fast colonizer, tolerates fluctuating water levels	
Lizard's Tail	2	Emergent	Yes	to 1 ft.	Yes	Low	Rapid growing,	

Plant Name Common (Latin)	Zone	Form	Avail-ability	Tolerance for Periodic Inundation	Native Plant	Wildlife Value	Special Requirements	Notes
Marsh Hibiscus (Hibiscus moscheutos)	2,3	Emergent	Yes	to 3 in.	Yes	Low		
Pickerelweed (Pontederia cordata)	2,3	Emergent	Yes	to 0.5-1.0 ft.	Yes	Low, ducks		
Pond Weed (Potamegaton)	2,3	Submergent	No	1.5-3.0 ft. deep	Yes	High, waterfowl, marsh and shore birds		
Rice Cutgrass (Leersia oryzoides)	2,3	Emergent	Yes	to 3 in. deep	Yes	Moderate, ducks, song-birds	Shade tolerant	
Sedges (Cyperus spp.)	2,3	Emergent	No	to 3 in. deep	Yes	Moderate, Waterfowl, song-birds		
Soft-stem Bulrush (Scirpus validus)	2,3	Emergent up to 10'	Yes	to 1.0 ft.	Yes	Moderate, good cover	Aggressive colonizer	
Smartweed (Polygonum spp.)	2	Emergent	Yes	to 1 ft. deep	Yes	High, waterfowl, song-birds	Fast colonizer	
Spatterdock (Nuphar luteum)	2	Emergent	No	to 1.5 ft. deep	Yes	Moderate, food, high cover	Fast colonizer, deals with fluctuating water levels	
Switchgrass (Panicum virgatum)	2,3,4 5,6	Perimeter emergent	Yes	to 3 in. deep	Yes	High, waterfowl, song-birds, game-birds	Tolerates wet/ dry conditions	
Sweet Flag (Acorus calamus)	2,3	Perimeter emergent 2-5'	Yes	to 3 in. deep	Yes	Low, for most spp., high for muskrat/beaver	Slow colonizer, tolerates drying	
Water Iris (Iris pseudoacorus)	2,3	Perimeter	Yes	to 3 in. deep	No	Low	Attractive, ornamental	
Water Cress (Nasturtium officinale)	Flowing water		No	to 6 in. deep	Yes	Moderate		
GRASSES/GROUND COVER:								
Bermudagrass (Cynodon dactylon)	4,5,6	Grass	Yes	Yes	No	Low	Erosion control, swales, useful for lawns, moderate maintenance	
Bristlegrass (fox tails) (Setaria spp.)	4,5,6	Grass	Yes	?	Some sp.	High		
Chewnings Red Fescue (Festuca comutata)	4,5,6	Grass	Yes	Yes	No	Moderate, ground feeding,	Erosion control, lawns, swales, shade tolerant, low to moderate	

(Table 9.2 cont.)

Plant Name Common (Latin)	Zone	Form	Avail-ability	Tolerance for Periodic Inundation	Native Plant	Wildlife Value	Special Requirements	Notes
						song-birds, browsers	maintenance	
Crownvetch (Coronilla varia)	6		Yes	No	No	Low	Excellent for stabilizing steep slopes, embankments, low or no maintenance, drought tolerant, erosion control, rapid growth, needs liming	
Kentucky Bluegrass (Pao pratensis)	5,6	Grass	Yes	Yes	No	Moderate	Lawns, swale, erosion control, handles traffic, moderate to high maintenance	
Kentucky "31" Tall Fescue (Festuca arundinacea)	4,5,6	Grass	Yes	Yes	No	Moderate	Low maintenance, lawns, swales, shade tolerant, rapid germination, erosion control	
Redtop (Agrostis alba)	3,4,5,6	Grass	Yes?	Yes	No?		Low to moderate maintenance, swales, shade intolerant, erosion control, wet soils	
Reed Canary Grass (Phalaria arundinacea)	3,4,5,6	Grass	Yes	Yes	No?	Low	Erosion control, swales, shade intolerant, not recommended for lawns, foot traffic	
Ryegrass, perrenial (Lolium perrene)	5,6	Grass	Yes	Yes	No?	Low	Erosion control, shade intolerant, lawns, moderate to high maintenance	
Switchgrass (Panicum virgatum)	5,6	Grass	Yes	Yes	No?	High		

SOURCES:

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Niering, William A. ed. 1984. Wetlands. Audubon Society Nature Guide Series. Chanticleer Press, New York, N.Y. 627 pp.

APPENDIX A

TECHNICAL DOCUMENTATION OF A SIMPLE METHOD FOR ESTIMATING URBAN STORM POLLUTANT EXPORT

This appendix describes the development of a Simple Method for estimating storm pollutant export delivered from urban development sites. The method was developed to provide an easy yet reasonably accurate means of predicting the change in pollutant loadings in response to development. This information is needed by planners and engineers to make rational nonpoint pollution decisions at the site level. The Appendix is organized as follows. In the first section, the Simple Method is derived in a step by step manner. Subsequent sections detail the technical analysis supporting each step.

1. Calculation of Storm Pollutant Export from the Site

The simple method is empirical in nature, and involves a two-step procedure. First, the mean concentration of the desired urban pollutant is obtained from Table A.1. Next, the runoff depth (R, in acre-feet) for the site is computed using the formula:

(EQ A.1): $R = [(P)(Pj)(Rv)/12]\ (A)$

where P = rainfall depth over the interval (inches)
Pj = fraction of rainfall events that produces runoff
Rv = mean runoff coefficient (i.e., proportion of rainfall converted to direct runoff)
A = watershed area (acres)
12 = conversion factor (inches to feet)

The computed runoff depth (R, in acre-feet) can be converted to an equivalent daily discharge (Q, in cubic feet/second/day), by:

(EQ A.2): $Q = (R)\ (43{,}560 \text{ sq ft})(\text{day}/24 \text{ hour})(\text{hr}/60 \text{ min})(\text{min}/60 \text{ sec})$

or, $Q = (R)(0.504)$

Given the mean concentration for a pollutant (C, mg/l or ppm) and the discharge rate (Q), the load over any interval (L, in pounds) is given by:

(EQ A.3): $L = (C)(Q)(5.39)$

where 5.39 is a conversion factor

By combining terms, the general equation for estimating urban runoff loads (expressed in pounds/acre/interval) is provided by:

(EQ A.4): $L = [(P)(Pj)(Rv)/12]\ (0.504)(C)(5.39)$

or more simply,

(EQ A.5): $L = [(P)(Pj)(Rv)/12]\ (C)(2.72)$

The user need only supply two parameters to estimate loads for any development site or condition; 1) the rainfall depth (P), and 2) site imperviousness (I, expressed as the fraction impervious area/total area). Pj, the fraction of precipitation (P) that produces any surface runoff is assumed to be a constant (0.9; see Section 7). The runoff coefficient, Rv, is assumed to be a linear function of percent imperviousness (I), specifically:

(EQ A.6): $Rv = 0.05 + 0.9\ (I)$

Flow-weighted mean concentrations (C) for each pollutant of interest can be found in Table A.1.

Table A.1: Average C Values For Stabilized Urban Sites

	TP	TSP	OP	TN	TKN	NH3	NO3	COD	BOD	ZN	PB
POLLUTANT "C" VALUE (mg/l)	0.26	0.16	0.12	2.0	1.5	0.26	0.5	35.6	5.1	0.04	0.02

2. Calculating Expected Pollutant Concentrations from the Site

The Simple Method can also provide the expected level of an urban pollutant for a specified frequency of occurrence or return interval. Estimates of this nature may be required to examine the frequency with which the runoff from an urban site might cause violations of a water quality standard for a receiving water. Because urban runoff data typically follows a log-normal probability distribution (EPA, 1983), the frequency that at which an urban pollutant exceeds a given concentration threshold can be calculated using the relationship:

(EQ A.7): $Cx = \exp\ [\ln X + (Zo)(\ln S).]$

where Cx = expected concentration at a given frequency of occurrence
lnX = mean of log-transformed data
Zo = the standard normal probability
lnS = standard deviation of the log-transformed data

Table A.2 summarizes urban pollutant levels for a series of common exceedance frequencies computed using EQ A.7 and the Washington area NURP dataset. Note that the exceedance frequency refers to the number of storms in which a given threshold level is exceeded. Based on the 1980-1985 National Airport record, approximately 65 storms generate measurable storm runoff in the Washington region each year.

Table A.2: Exceedance Frequency For Selected Urban Pollutants

POLLUTANT CONCENTRATION (mg/l)	PERCENT OF STORMS IN WHICH GIVEN CONCENTRATION VALUE IS EQUALLED OR EXCEEDED				
	50%	25%	10%	5%	1%
Sediment	31.	71.	151.	235.	545.
Total Phosphorus	0.27	0.43	0.65	0.82	1.31
Total Nitrogen	2.2	3.2	4.5	5.6	8.2
COD	42.	61.	84.	103.	149.
Lead	0.02	0.04	0.08	0.11	0.15
Copper	0.01	0.02	0.04	0.06	0.11
Zinc	0.06	0.10	0.16	0.22	0.36

3. Derivation of Mean Urban Runoff Concentrations-(C)

Over 300 storm runoff events were sampled at eight sites dispersed throughout the Washington metropolitan area during the 1980-1981 NURP study (MWCOG, 1983b). The sites included a wide range of soils, slopes and land uses, and ranged from 10 to 90% impervious cover. Automated samplers collected a composite sample during each runoff event that effectively represented the average pollutant concentration. Statistical studies on both the local and national NURP data have indicated that urban runoff and pollutant levels follow a log-normal distribution, which greatly facilitates the analysis of highly variable data (Driscoll,1986; NVPDC, 1983).

The procedure followed in this study was as follows: 1) Log-transformed event mean concentration values (EMC) were calculated for each storm at each site; 2) The EMC's at each site were weighted by the corresponding depth of runoff volume for the storm event (in inches/acre), and 3) relevant statistics (means, medians and standard deviations) were computed for flow-weighted EMCs for both individual sites and all sites lumped together.

F-tests were used to establish whether the variance for individual sites was significantly different from the variance of the entire group. With the exception of two sites, the variances were found to be statistically similar. The exceptions occurred at the sites with the smallest number of storms sampled. Therefore, it was concluded that the individual site sample populations could be compared with each other and with the entire lumped population.

A Students t-test was then applied to test whether the site means and the grand mean were significantly different (alpha = 0.05, two-tailed test). The results of the t-test analysis are shown in Table A.3. For most of the parameters of interest, there was no statistically significant difference between the individual site means and the grand mean, although some exceptions to this rule were evident at a few sites and for suspended sediment. Taken as a whole, the analysis indicates that the average pollutant concentration (EMC) of all the Washington NURP sites is a reasonably reliable predictor of the levels encountered at any individual site. Moreover, these results suggest that there were no consistent relationship between pollutant concentrations and land use. Other statistical analyses of national urban and highway runoff datasets (EPA, 1983; Shelley and Garboury,1986) also failed to find consistent trends between pollutant concentration levels and watershed land-use (or imperviousness). These findings imply that differences in pollutant loads among urban land-uses is due primarily to greater storm runoff volume than to enhanced pollutant concentrations (Athayde,1986).

Table A.3: Statistical Comparison of Urban EMCs at Washington NURP Sites

STORM EMC (mg/l)	Metro Washington NURP Sites								ALL SITES (n=298)
	BURKE (n=60)	DUFEF (n=8)	FARDGE (n=50)	FAIROK (n=11)	LAKER (n=49)	STED (n=47)	STRAT (n=32)	WEST (n=41)	
TOTAL N	1.81	2.39	2.84*	1.55	2.13	2.02	2.07	1.97	2.00
NO3	0.51	0.41	0.73*	0.34	0.63	0.54	0.30	0.48	0.48
NH3	0.21	0.25	0.43*	0.14	0.27	0.28	0.26	0.23	0.26
Org-N	1.06	1.73	1.69*	1.07	1.22	1.20	1.51	1.16	1.25
TOTAL P	0.19	0.51	0.38	0.20	0.33	0.31	0.30	0.30	0.26
OP	0.09	0.24	0.19*	0.03	0.07*	0.15	0.16	0.14	0.12
TSP	0.13	0.38	0.26*	0.07	0.08*	0.20	0.21	0.19	0.16
Org-P	0.06*	0.13	0.12	0.13	0.21*	0.11	0.09	0.11	0.10
COD	25.0*	54.5	49.4	37.6	53.5*	33.7	41.3	40.3	35.6
BOD-5	3.6	7.8	7.0	5.4	7.6	4.8	6.0	5.8	5.0
BOD-20	5.0	10.9	9.9	7.3	10.6	6.7	8.2	8.1	7.3
TSS	10.9*	29.7	13.*	81.*	162.*	37.8*	39.0*	44.0*	25.8
Lead (ug/l)	11.4	5.6	6.2	29.	51.*	32.	20.	21.	18.0
Zinc (ug/l)	29.	146.*	77.*	80.	77.*	66.*	59.	30.	37.

NOTES: (*) denotes a significant difference between the individual site mean and the grand mean, using a two-tailed Students t-test and a 95% level of significance.

(a) COD:BOD-20 and COD:BOD-5 rations of 5:1 and 7:1, respectively, were assumed from data in US EPA (1983).

Further, the national NURP data analysis demonstrated that urban pollutant concentrations were not significantly correlated with either storm event runoff volume or storm intensity at over two-thirds of the sites examined (US EPA,1983). Had stronger correlations between runoff volume and pollutant concentrations been observed (i.e, if C was related to runoff volume), then the use of a mean C value would introduce serious bias to the Simple Method (which assumes that runoff volume and pollutant concentration are independent of each other). The minor bias that does exist in the Washington NURP data (primarily as a result of dilution at high flows) was partially mitigated by flow-weighting individual storm EMCs (i.e., small storm events contributed proportionally less to the mean C value than large storms events, that deliver the greatest proportion of the annual load).

4. Predicting Suspended Sediment Levels

Suspended sediment was the only major urban pollutant which departed from these general statistical properties. Individual site means and variances were significantly different from each other and the grand mean for nearly every site (Table A.3). Further, no significant correlations between sediment concentration and runoff volume or watershed imperviousness were found at the Washington sites, and very few were found in the national NURP data (US EPA, 1983). Due to its highly variable behavior from storm to storm and site to site, sediment loads cannot be predicted on the basis of a grand mean EMC. The only quasi-predictive behavior associated with suspended sediment in urban areas is that it appears to be generally related to watershed size. As shown in Figure A.1, mean storm sediment concentrations tend to increase with drainage area in 25 urban watersheds in the Washington region, ranging from 5 to over 100,000 acres in area (data sources: OWML, 1983, Hickman, 1984, MWCOG, 1983b, NVPDC, 1978).

The higher storm sediment levels observed in larger urban watersheds is primarily due to bank and channel erosion, rather than erosion of pervious areas by overland flow, or washoff of sediments from impervious areas within the watersheds. Under this theory, as watershed size becomes larger, the length of the stream channel network and the susceptibility to channel erosion increases markedly. Most small headwater streams in the Washington area have abundant supplies of sediment in storage, which has been gradually deposited by previous centuries of agricultural erosion, or more recently, by construction-related erosion (Meade,1982: Costa, 1975; Wolman and Schick, 1967). The large quantities of sediment in channel storage are gradually resuspended and transported out of the watershed, by the increased peak and frequency of floods which follow urbanization.

Channel erosion appears to be the most feasible source of sediment, since alternative sources of sediment (i.e., pervious area erosion and/or impervious area washoff) do not appear adequate to sustain high sediment levels observed during storms. Erosion of pervious areas in most stabilized urban sites is minimized by the extensive cover of lawns and open space, and washoff from impervious areas is limited by the atmospheric supply of solids, which amounts to less than a tenth of a ton per year (see Table 7.2). Both sources appear sufficient to maintain the base level of suspended sediment concentrations of about 15-25 mg/l in storm runoff.

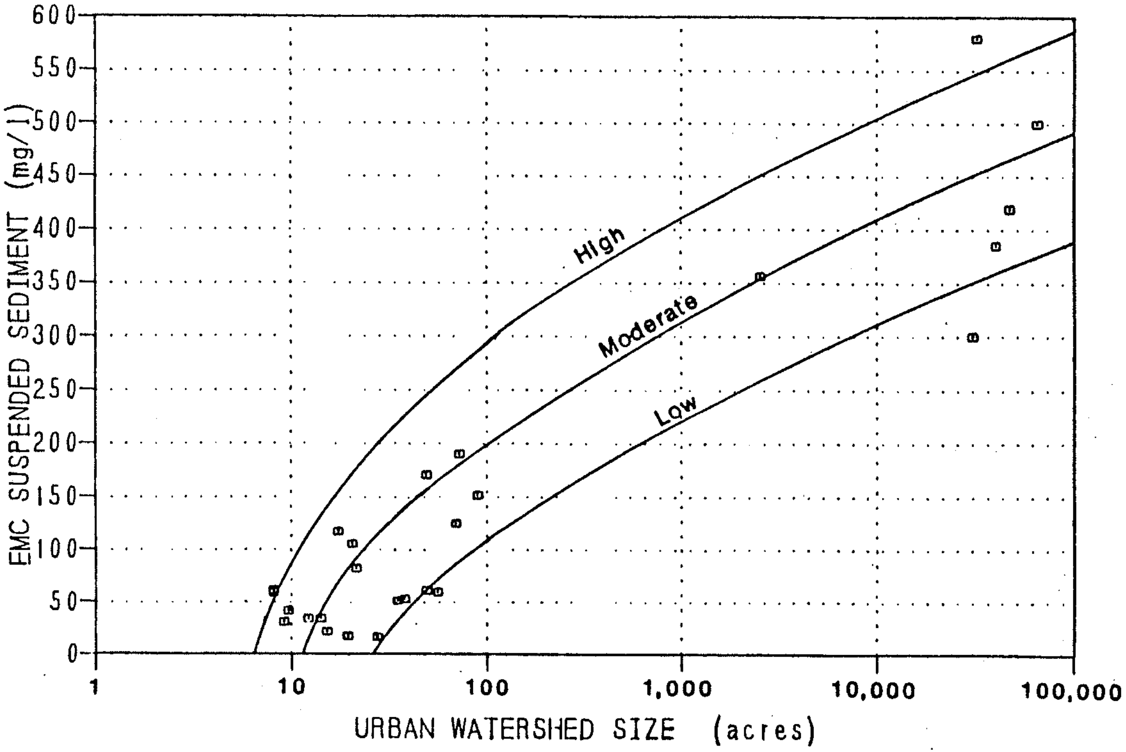
The relationship between mean storm sediment levels and drainage area is offered as a a first-cut estimate of the expected storm sediment concentrations for specific development situations. It is recognized that this semi-log approach has limited predictive capability, and it is hoped that better, more deterministic models of urban erosion and sediment transport can be developed to replace it.

A rather wide envelope has been drawn around the sediment EMC values in Figure A.1 to reflect the considerable variation observed in the field. The choice of a high, moderate or low value curve is a matter of subjective interpretation, but the following criteria are offered as guidance:

Table A.4 Watershed Channel Network Conditions

	LOW EMC	MODERATE EMC	HIGH EMC
Stability condition of channel	Vegetated swales or storm sewers	Intermediate	Open channel, cut banks alternating w/channel sandbars, fallen trees
Channel sediment storage	Small deposits in storm drains, stabilized land use	"	Large silt or clay deposits evidence of recent or ongoing construction. Water becomes murky after disturbing bottom
Stream velocity	Low slope, low imperviousness	"	High slope, high imperviousness

Figure A.1: Storm Sediment EMC's As a Function of Watershed Size



5. Urban Pollutant Concentrations for Use in Special Conditions

The mean urban pollutant levels derived in the Washington NURP study (Table A.1) may not always be appropriate for every development situation. The Washington NURP sites were located in stabilized, recently suburbanized areas, and should generally represent the pollutant levels emanating from new development sites in the region.

However, older, poorly maintained urban neighborhoods generate significantly higher urban pollutant levels. The average pollutant levels for five older residential catchments monitored in downtown Baltimore, Maryland (BRPC, 1986) are provided in Table A.5. As can be seen, the log-mean EMCs for downtown Baltimore are from 2 to 5 times higher than those reported for suburban Washington. The higher pollutant levels reported at the Baltimore sites were attributed to poor urban "housekeeping" (e.g., poor trash removal, accumulation of debris, deteriorating housing stock, high traffic volumes, poor upkeep of lawns, and more mature vegetation). If older residential areas are being evaluated, it is recommended that the Baltimore data should be used instead of the Washington data.

Similarly, pollutant levels measured in highly imperviousness central business districts (CBDs) are often higher than suburban residential areas. This is due in part to greater traffic volume and/or higher atmospheric loading rates (MWCOG, 1983b). Pollutant levels monitored in the central business corridor of downtown Washington, DC are provided in Table A.5. Nutrient, BOD and trace metal levels measured during 27 storms at two sites were often higher than other residential or commercial suburban sites monitored in the Washington NURP study (NVPDC, 1981;MWCOG, 1983b).

For comparative purposes, flow-weighted EMCs for forested areas are also provided in Table A.5. These values were derived from a two year monitoring study of several small forested watersheds in the Occoquan basin during 1980-81 (OWML, 1982), and can be used to roughly estimate "natural" background loadings contributed by undeveloped areas. Although these forest C values may not always hold for all pre-development conditions, they serve as a reference point to quantify the increase in pollutant export following urban development activity.

Highway runoff has been monitored by the Federal Highway Administration for nearly 300 storm events at eight urban highway sites across the nation (Shelley and Gaboury, 1986). Pollutant EMCs tend to be higher than most typical urban areas, particularly for metals and orthophosphate, which is thought to be due to vehicular emissions.

Finally, Table A.5 includes the national NURP event mean concentrations which were obtained from over 2300 storms monitored at 22 project sites across the nation (US EPA, 1983). Since most of the C values in Table A.5 are specific to the Baltimore-Washington area, it is recommended that the national NURP C values be used for areas outside of the Mid-Atlantic states. When the mean values from the local and national NURP studies are compared, it is evident that Washington area urban runoff has slightly lower urban pollutant concentrations (US EPA,1983).

The Simple Method has been designed such that any urban storm monitoring dataset can be used as a basis for generating loads. Thus, if a future monitoring effort indicates that mean pollutant levels have changed, or are different in a specific development situation, the simple method can be easily modified.

Table A.5: Urban 'C' Values For Use Special Conditions(mg/l)

POLLUTANT	NEW SUBURBAN NURP SITES (Wash.,DC)	OLDER URBAN AREAS (Baltimore)	CENTRAL BUSINESS DISTRICT (Wash.,DC)	NATIONAL NURP STUDY AVERAGE	HARDWOOD FOREST (Northern Virginia)	NATIONAL URBAN HIGHWAY RUNOFF
PHOSPHORUS						
Total	0.26	1.08	-	0.46	0.15	-
Ortho	0.12	0.26	1.01	-	0.02	-
Soluble	0.16	-	-	0.16	0.04	0.59
Organic	0.10	0.82	-	0.13	0.11	-
NITROGEN						
Total	2.00	13.6	2.17	3.31	0.78	-
Nitrate	0.48	8.9	0.84	0.96	0.17	-
Ammonia	0.26	1.1	-	-	0.07	-
Organic	1.25	-	-	-	0.54	-
TKN	1.51	7.2	1.49	2.35	0.61	2.72
COD	35.6	163.0	-	90.8	>40.0	124
BOD (5-day)	5.1	-	36.0	11.9	-	-
METALS						
Zinc	0.037	0.397	0.250	0.176	-	0.380
Lead	0.018	0.389	0.370	0.180	-	0.550
Copper	-	0.105	-	0.047	-	-

6. Derivation of the Runoff Coefficient (Rv)

The runoff coefficient (Rv) is a useful measure of site response to rainfall events, and is simply calculated as:

(EQ A.8): $Rv = r/p$

where r and p are the volume of runoff and rainfall, respectively, expressed in watershed inches

The dimensionless number represent the extent to which rainfall is translated into surface runoff, and varies according to watershed soils, slopes, cover and urbanization. Driscoll (1983) has computed mean and median Rv's at over 50 sites monitored in 16 NURP projects around the nation, and found that most of the variation in mean Rv among sites can be attributed to differences in the level of urbanization, and in particular, to the extent of site imperviousness. Rv's were found to be relatively consistent at individual sites, and were only weakly correlated with storm-related variables such as precipitation volume, intensity and duration. Driscoll concluded that the runoff coefficient could serve as a reliable estimator of runoff volumes, given an initial estimate of rainfall volume.

A summary of the mean and median Rv's for 44 small watershed sites is provided in Table A.6. The table includes Rv's for four Washington NURP sites not included in Driscoll's original analysis. Seven sites were omitted from further analysis due to small sample size. Linear regression analysis was conducted, using the site mean Rv as the dependent variable and watershed imperviousness (I) as the independent variable. The resulting equation was:

(EQ A.9) $Rv = 0.05 + .009\ (I)$ (adj. $r2= 0.71$)

It should be clearly noted that Equation A.9 only predicts storm runoff volumes. It does not predict the baseflow component of annual runoff volume. For most urban sites, this distinction is not important. However, in large, low density residential watersheds, baseflow can be an important component of the annual runoff volume. Figure A.2 shows the difference in the value of the storm and annual runoff Rv's as a function of watershed imperviousness. As can be seen, baseflow dominates the water balance for a site when total imperviousness is less than 10%. As a practical matter, however, baseflow does not often appear in most small developments (i.e., less than 25 acres). Therefore, its contribution to annual runoff volume can usually be ignored. In addition, local research has shown that pollutant levels found in urban baseflow can seldom be distinguished from normal "background" levels (NVPDC,1979). This suggests that urban baseflow will not deliver extra pollutant loadings to receiving waters.

If for some reason, however, it is necessary to compute an annual mass balance of runoff or pollutants from a site, the additional flow and pollutant loads carried in baseflow can be easily calculated, given a knowledge of baseflow quantity and quality. Baseflow quantity can be determined from either an analysis of USGS dry weather discharge records for the watershed, or by the difference between the annual and storm runoff coefficients shown in Figure A.2. Baseflow pollutant concentrations can be inferred from regional or local dry-weather water quality monitoring data. The baseflow pollutant load can then be calculated as:

(EQ A.10) $L= [(P)(Rva)-(P)(Pj)(Rv)]/12\ [(Cb)(A)(2.72)]$

where Rva= Annual runoff coefficient (From Figure A.2)
Cb = average dry-weather pollutant concentration (mg/l)
and all other parameters as defined previously

Correction for baseflow discharge/concentrations should provide acceptable loading estimates, provided no other pollutant sources exist in the watershed (e.g., municipal sewage treatment discharges, livestock operations, or agricultural activities).

7. Derivation of Correction Factor (Pj)

The Rv obtained using equation A.9 needs to be adjusted to eliminate the portion of annual rainfall which does not produce any direct runoff. As Figure A.3 shows, over 50% of all storm events recorded at the National Airport weather station between 1981 and 1985 had less than 0.2 inches of total rainfall. The rainfall from minor storms may be entirely stored in surface depressions and eventually lost by evaporation or infiltration. As a result, no runoff is produced.

The extent of these losses was evaluated by double mass curve analysis, in which the cumulative volume of rainfall in selected intensity classes were compared for the National Airport and the Washington NURP rainfall gages. Since the NURP rainfall data was only stored if there was a corresponding runoff event for the site, the mass curve analysis indicates whether any rainfall intensity classes were under-represented in comparison to the National Airport data. The corresponding volume of "missing" rainfall at the NURP gage was assumed to represent the rainfall events that produced no appreciable runoff. While the distribution of rainfall in the higher rainfall intensity classes was similar for both the National Airport and the NURP rainfall gages, smaller intensity events were distinctly underrepresented at the NURP gages. The amount of "missing" rainfall calculated by difference is shown in Figure A.4. Based on this analysis, it is suggested that about 10% of the annual rainfall volume is so slight that no appreciable runoff is produced, therefore the value of Pj in Equation A.1 can be set to 0.9.

Figure A.2: Difference Between Storm Rv and Annual Runoff Rv

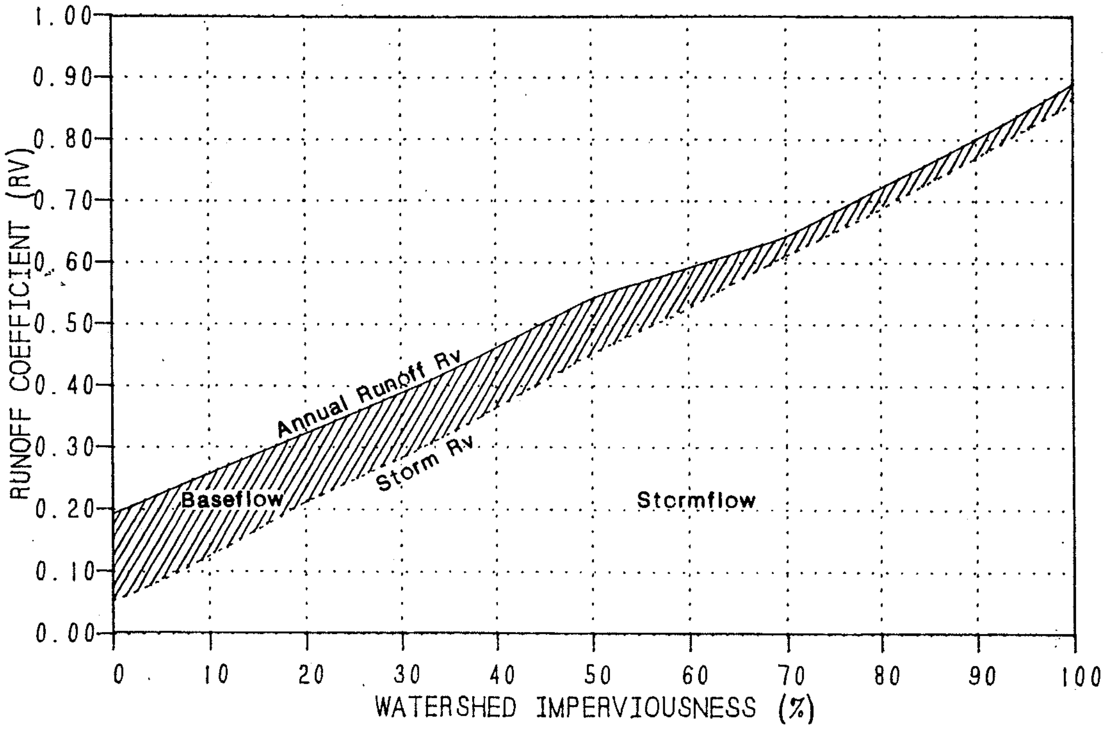


Table A.6: Calculated Storm Rv's For 44 Small Urban Catchments

OBS	NURP PROJECT	SITE NAME	PERCENT IMPERVIOUS	MEAN Rv (a)	MEDIAN Rv	COEFFICIENT OF VARIATION
1	CO1	Big Dry	41.	0.35	0.32	0.46
2	CO2	Cherry	38.	0.18	0.16	0.45
3	CO3	Claude	24.	0.16	0.15	0.40
4	DC1	Burke	33.	0.46 (b)	0.53	.
5	DC2	Laker	33.	0.25	0.18	.
6	IL1	John N	19.	0.19	0.15	0.73
7	MD1	Homewd	29.	0.47	0.38	0.73
8	MD2	Mt Wash	29.	0.24	0.12	1.76
9	MD3	Res Hill	76.	0.56	0.49	0.54
10	NY1	Carrol R	20.	0.24	0.20	0.65
11	NY2	Cranston	22.	0.17	0.16	0.33
12	NY3	E Roch	38.	0.22	0.20	0.42
13	WA1	Surrey	29.	0.20	0.17	0.63
14	WI1	Burbank	50.	0.37	0.27	0.92
15	WI2	Lincoln	57.	0.43	0.38	0.55
16	DC3	Westlgh	21.	0.17	0.13	.
17	IL2	John S	18.	0.18	0.16	0.47
18	WA2	Lakehill	37.	0.24	0.20	0.62
19	IL3	Mattis S	37.	0.37	0.30	0.73
20	CO4	Asbury	22.	0.99	0.19	0.97
21	IL5	Comb Inl	17.	0.19	0.17	0.48
22	NC1	1023	27.	0.11	0.09	0.69
23	MA1	Jordan	21.	0.26	0.22	0.65
24	DC4	Stedwk	34.	0.28	0.20	.
25	IL6	Mattis N.	58.	0.73	0.63	0.57
26	WI3	Wood Ctr.	81.	0.82	0.76	0.42
27	MA4	Rt 9	23.	0.28	0.20	0.99
28	NY4	Cedar	5.	0.11	0.08	1.05
29	MA7	Tilley Br	6.	0.02	0.01	1.17
30	MA8	Addison	69.	0.65	0.58	0.53
31	CA2	Comml	99.	0.98	0.98	0.04
32	CO4	Villa 1	91.	0.99	0.93	0.45
33	NC2	1013	69.	0.90	0.84	0.38
34	NY7	South GTE	21.	0.21	0.20	0.28
35	WI5	Post Off	99.	0.92	0.90	0.19
36	NH1	Pkg Lot	90.	0.74	0.66	0.50
37	NY10	Thornell	4.	0.08	0.06	0.93
38	NY12	West Br.	1.	0.12	0.07	1.44
39	NY13	Thomas	11.	0.05	0.04	0.56
40	NY14	Sherrif	7.	0.08	0.05	1.25
41	DC5	Dandrg	55.*	0.57	0.52	.
42	DC6	Fairrdge	34.*	0.47	0.37	.
43	DC7	Fairoak	90.*	0.75	0.81	.
44	DC8	Stratwd	22.*	0.42	0.38	.

Notes: (*) new sites not in Driscoll (1983)
(a) mean Rvs calculated as: (median) [SQRT (1 +Cv**2).]
(b) Burke recomputed to exclude baseflow interference

Figure A.3: Rainfall Frequency Distribution at National Airport-1980 to 1985

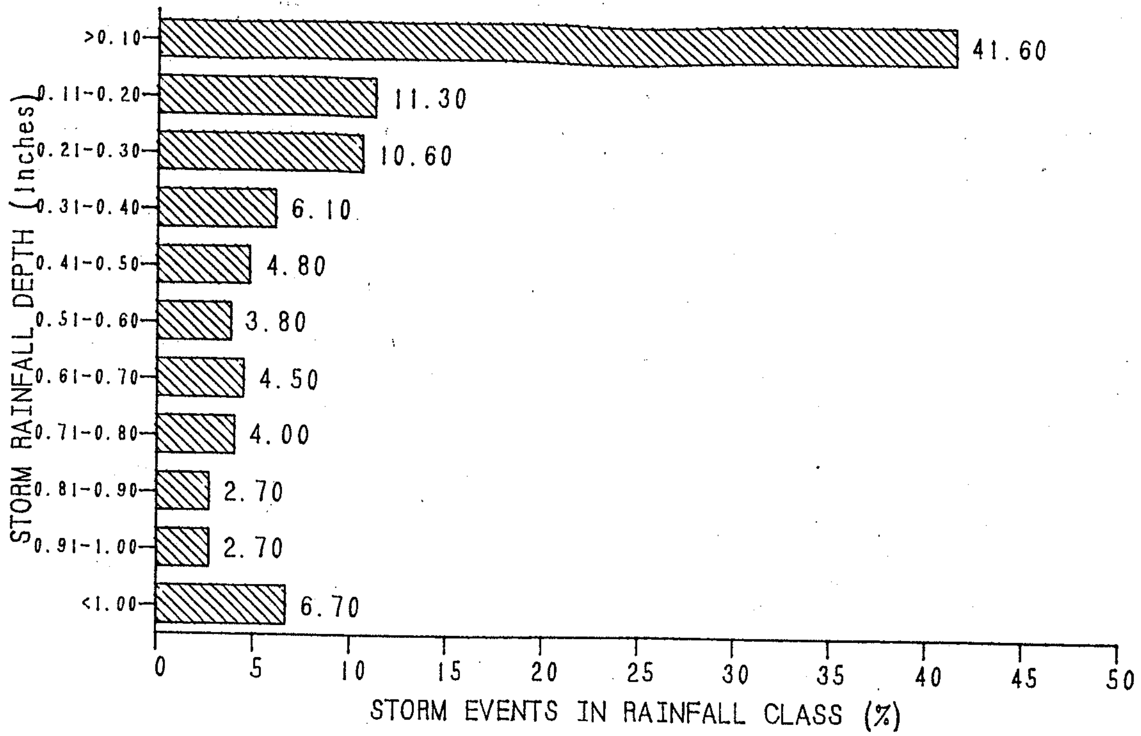
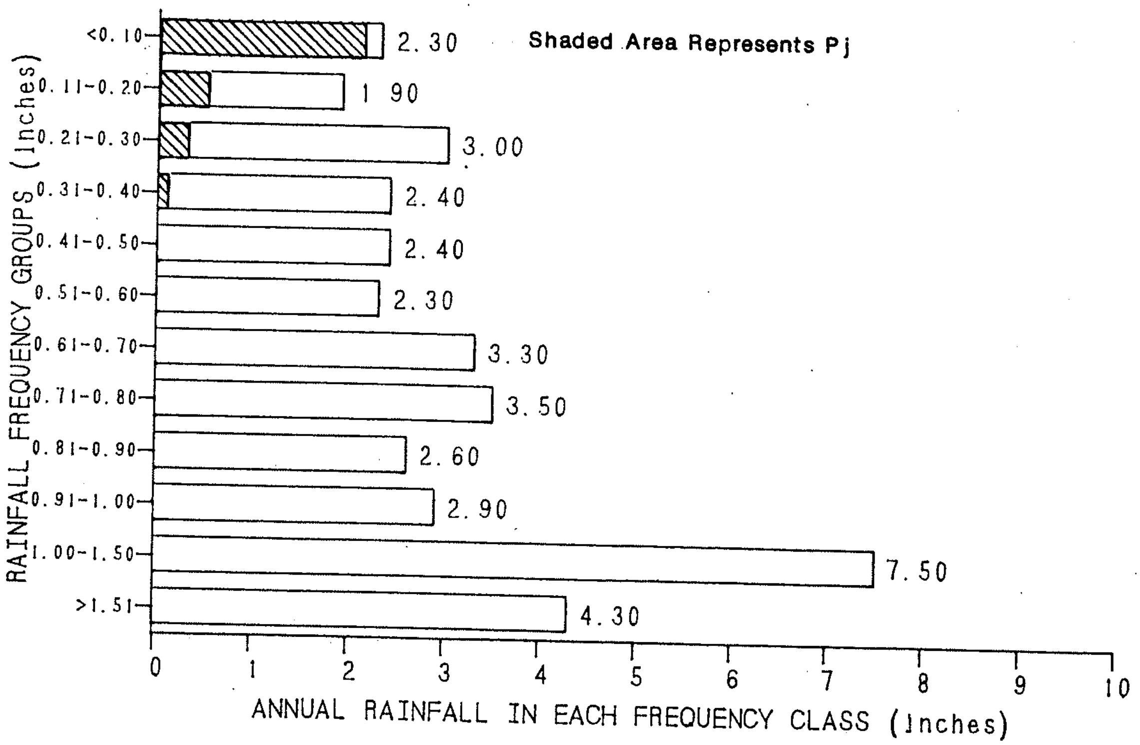


Figure A.4: Fraction of Annual Rainfall Events That Produce No Runoff



NOTE: Frequency of annual rainfall in each depth class; Five year running average at National Airport, Arlington, Virginia.

APPENDIX B: BANKFULL FLOODING FREQUENCY ANALYSIS

General Approach

It is generally acknowledged that the 1.5 to 2 year return frequency storm controls the shape and form of natural channels (Leopold et al.,1964; Anderson, 1970). This finding has been incorporated into local SWM policies by requiring the post-development increase in the magnitude of the two year or bankfull flood be controlled to pre-development rates. However, by itself, this policy is insufficient to adequately control downstream bank erosion. This is because watershed development tends to increase the frequency of bankfull flooding, in addition to increasing the magnitude of flooding. The increased number of bankfull floods, which are an erosive condition, raises in turn, the potential for increased streambank and channel erosion.

For example, conventional detention ponds designed to keep post-development discharge within the banks (i.e., the pre-development two year flood level) may still experience two to ten bankfull floods each year, instead of one every other year. Consequently, the abundant quantity of sediment stored in the channel and banks is subject to a longer interval of erosive conditions during the year. To reduce the potential for erosion, pond designs should attempt to control runoff so that the natural frequency of the pre-development bankfull flooding is preserved (i.e., store enough runoff volume to maintain the post-development bankfull flooding frequency to approximately one event every two years, on average).

This appendix describes a preliminary analysis of the effect of watershed development on bankfull flooding frequency in small watersheds. Next, it explores the question of how much extended storage is needed to mimic the pre-development bankfull flooding frequency. Both analyses utilize relatively simple hydrologic models and require several simplifying assumptions. Thus, the results should be viewed with some caution, until they have been corroborated with field data.

Methodology

To perform a frequency analysis of bankfull flooding events, the following hydrological variables must be predicted over an extended rainfall time-series.

1. The natural pre-development peak discharge rate (Qp) and associated runoff volume (Qvp) for the two year design storm, for different sized watersheds (denoted by the watershed time of concentration:Tc)

2. Post-development peak discharge rates (Qpd) and associated runoff volumes (Qvd) under expected levels of watershed development (expressed in terms of percent imperviousness).

3. The frequency that post-development peak discharge rates (Qpd) equal or exceed the pre-development peak discharge rate (Qp) as a function of I, for a specified Tc.

4. The sizing rules used to define extended detention runoff storage. This requires the use of a runoff volume coefficient (C').

5. The uncontrolled runoff volume (Xvol) which is not removed from the post-development storm hydrograph by extended detention storage (Xst). Xvol is then compared to the pre-development bankfull flood runoff volume (Qvp) to assess whether or not Xst is sufficient to control bankfull flooding.

6. The change in frequency in bankfull flooding that can be attributed to extended detention storage, in comparison to an uncontrolled situation.

The Rational method was used to calculate peak discharge rates (Qp and Qpd), as it appears to be more reliable than current SCS methods for predicting peak discharge rates in small watersheds for relatively small storms (which are the major focus of this analysis). Pre-development peak discharge rates (Qp) were computed using regional rainfall intensity data and using a runoff coefficient for forested conditions. Post-development peak discharge rates (Qpd) were then computed with the Rational Formula, after calculating an adjusted peak runoff coefficient (C), based on the degree of watershed imperviousness (5% increments). A six-year record of short-term maximum monthly rainfall intensity values (Greenbelt, Maryland NWS weather station) provided input values for rainfall intensity. The time-series of storms was assumed to be representative of the normal distribution of storms expected in the Washington area.

The Rational formula was then solved for each of the 72 storms in the rainfall intensity record to create a corresponding time-series of estimated post-development peak discharge rates (Qpd). To extend the results, the analysis was performed for watershed concentration times (Tc) ranging from 15 minutes to two hours. The Qpd rates computed over the six year record were then compared to the reference pre-development peak discharge rate (Qp). If a Qpd rate equalled or exceeded the pre-development Qp rate, it was considered to be a bankfull flood. The frequency of post-development bankfull flooding was then tabulated for 5% increments of watershed imperviousness.

To evaluate the impact of extended detention storage on the frequency of the pre-development bankfull flood, the same analysis was repeated, except that a fixed volume of extended detention storage (Xst) was specified (0.25 to 3.00 inches of rainfall*C'), and subtracted from the total runoff volume (Qvd) generated from each storm. This reflects the fact that extended detention storage will be discharged well after the peak of the initial storm event, and is in practical terms, removed from storm hydrograph.

If the remaining storm runoff volume (Xvol) was less than 60% of the runoff volume generated by the pre-development two-year storm, then the storm was considered to be adequately controlled. If not, the storm was not effectively controlled. Graphs were then constructed that compared the frequency of the pre development 2 year flood under each extended detention rule, and without any extra detention at all. The details and assumptions of the analysis are described step by step below:

STEP 1. Compute Pre-Development Peak Discharge Rate and Runoff Volume

The Rational formula computes peak discharge (Qp) as:

(EQ B.1): $Qp= (C)(i)(A)$

where Qp= instantaneous peak discharge, in cfs
C = runoff coefficient

i = rainfall intensity for a duration equal to the watershed time of concentration, in/hr

A = watershed area, acres

Values for i were obtained from the Baltimore intensity-duration-frequency (idf) curve for the two-year return storm, for watershed time of concentrations of 15, 30 and 45 minutes, and 1 and 2 hours. Area (A) is the same for both pre- and post-development conditions, and therefore was ignored (i.e., Qp is for a unit area). A pre-development, natural condition runoff coefficient (C) of 0.15 was taken from an expanded table of C values supplied by McCuen (1986), which corresponds to a forest situated on moderately sloping B soils.

The runoff volume for the pre-development storm was computed as:

(EQ B.2): Qvp= 0.9*Tc*Qp

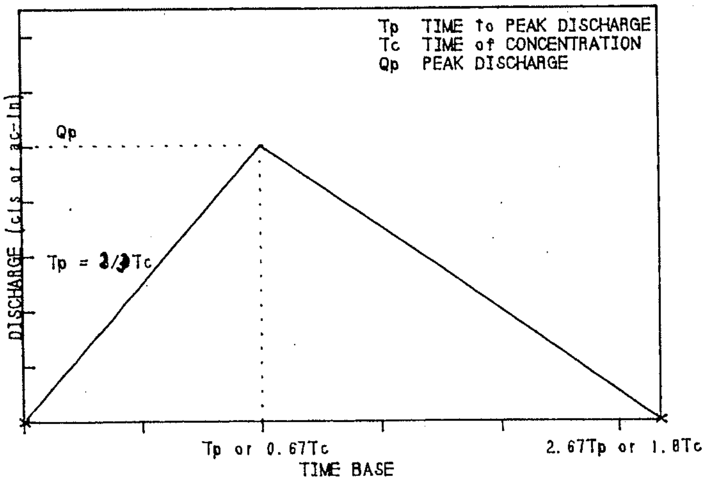
where: Qvp= runoff volume in cfs/hr

Tc= watershed time of concentration (hrs)

Qp= two-year peak discharge calculated above (cfs)

The SCS triangular hydrograph was assumed to reasonably portray runoff volumes. It has a time-base equivalent to 1.8 times the watershed time of concentration. Given the peak discharge (Qp), the total runoff volume is equal to 1/2 (base)(height), as shown in Figure B.1 below:

Figure B.1: Relationship Among Hydrological Variables in the SCS Triangular Unit Hydrograph



It can also be shown that for Qvp (and Qvd), the units of cfs-hr and watershed inches are interchangeable, when A is held constant.

one acre-inch = (0.083 ft/inch)(43,560 ft2/acre)/(60 min/hr)(60sec/min)

= 1.0083 (ft3)(hr)/sec , or approximately

(EQ B.3): ac-in = cfs-hr = Qv

STEP 2. Compute Post-Development Peak Discharge Rate/Runoff Volume

The analysis in STEP 1 was repeated using values of C based on watershed imperviousness in increments of 5%. This was done by developing an equation that computes post-development C as a function of imperviousness:

(EQ B.4): $C = (I)(X1) + (P)(X2)$

where: I= percent watershed imperviousness
P= percent watershed perviousness (1-I)
X1= runoff coefficient for impervious areas
X2= runoff coefficient for pervious areas

Constant values of X1 and X2 were obtained from an expanded table of runoff coefficients developed by McCuen (1986). X1 was set to 0.87, which corresponds to a completely impervious surface (such as a parking lot), and X2 was set to 0.15 (corresponding to a forest situated on moderately sloping B soils). C values were then computed in 5% increments from 0 to 100% imperviousness using EQ. B.4. Post-development peak discharge (Qpd) could then be calculated using EQ. B.1. It should be noted that equation B.4. is quite simplistic in that it does not explicitly consider variations in soil, slope and antecedent moisture conditions, nor does it vary in response to different rainfall intensities.

A second runoff coefficient is needed to estimate post-development runoff volume (Qvp) associated with the rainfall time-series. This coefficient (C') can be defined as follows:

(EQ B.5): $C' = r/p$

where: r = runoff depth (inches)
p = rainfall depth (inches)

By previous definition, it can be shown that:

(EQ B.6): $r = Qv$

(EQ B.7): $p = (Tc)(i)$

Thus, by combining equations B.1 and B.2: under the assumption of unit area:

$Qv = (0.9)(tc)(C)(i)$ and substituting r and p terms,

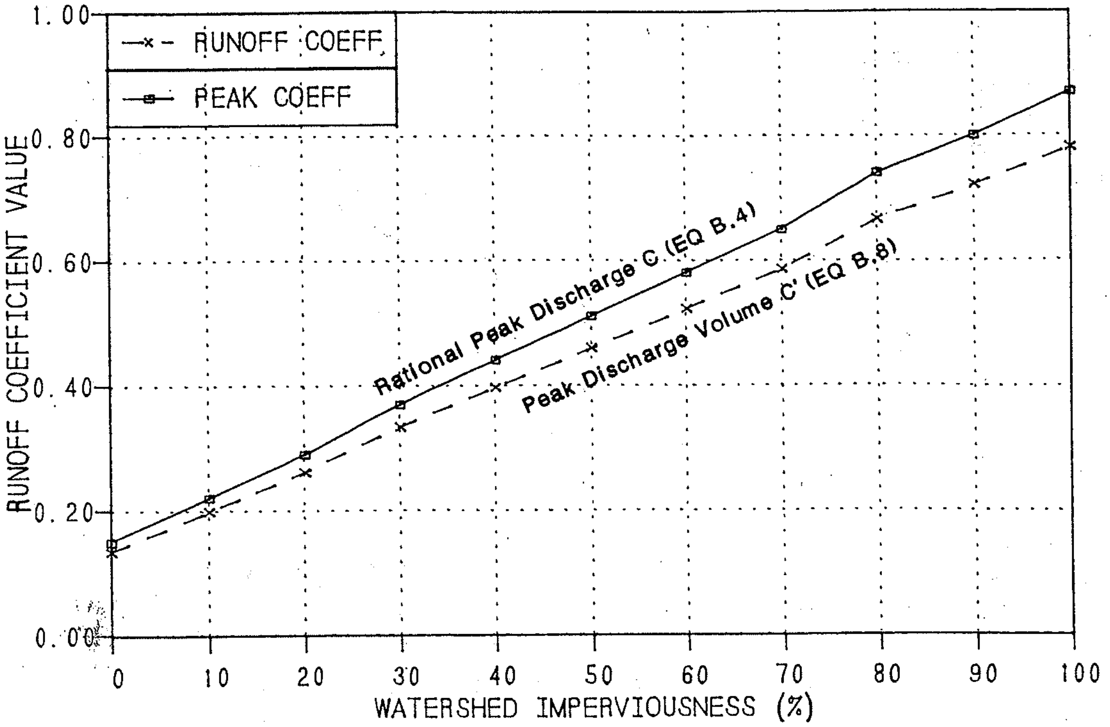
$r = (0.9)(p)(C)$ dividing both sides by p,

$r/p = 0.9*C$ and since $r/p = C'$, then:

(EQ B.8): $C' = 0.9*C$

Figure B.2 shows the relationship between the Rational or peak discharge C and the storm runoff volume C', as a function of percent I.

Figure B.2: Relationship between Peak Discharge and Storm Runoff Coefficients

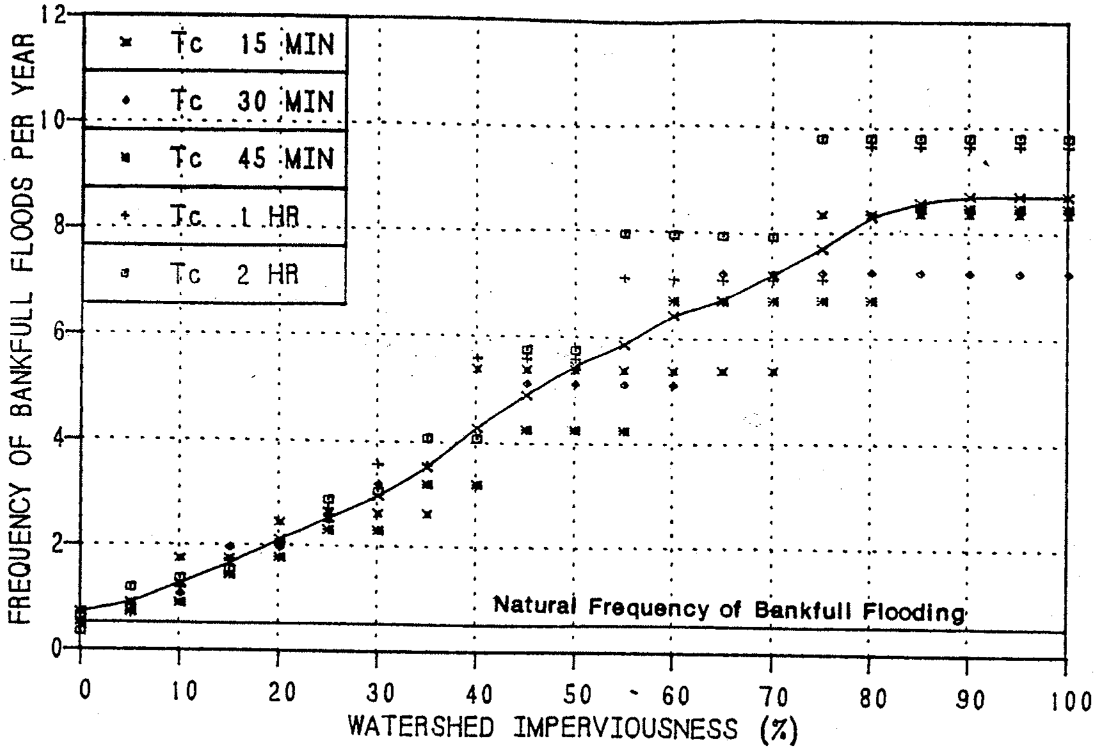


Step 3: Compute Increased Frequency of Bankfull Pre-development Floods

The frequency of bankfull flooding as a function of watershed imperviousness (I) was estimated using a six-year time series of monthly maximum precipitation values from Greenbelt, Md. contained in the NWS publication Hourly Precipitation Data for Maryland. This station was selected because of its close proximity to the metropolitan area and the fact that it records maximum rainfall intensity in relatively short time increments (15, 30, 45, 60 and 120 minutes) A monthly maximum rainfall intensity is reported for each time increment over a 72 month period. This record was used to approximate the actual or expected rainfall intensity over a long time interval.

A computer program was developed (Brown, 1987) that computes the peak discharge (Qpd) for each of the 72 maximum monthly rainfall intensity values, as a function of both different levels of development (0 to 100% imperviousness, in 5% increments), and different watershed times of concentration (15 minutes to 2 hours). If the post-development peak discharge rate (Qpd: computed in Step 2) was equal to or greater than the pre-development bankfull flood (Qp: obtained in Step 1), then the rainfall event was flagged as a bankfull flood (since it is assumed a flood greater than the pre-development bankfull flood would be controlled by the 2 year orifice/weir in a conventional detention pond). Annual bankfull frequency values as a function of watershed development are shown in Figure B.3. Since only moderate variation in frequency was observed for different times of concentration, a single curve, averaged for all times of concentration, was drawn to relate flooding frequency to watershed development.

Figure B.3: Frequency of Bankfull Flooding As a Function of I



Step 4: Define Storage Rules For Extended Detention

Eight different extended detention sizing rules were evaluated to determine their impact on bankfull flooding frequency under different levels of watershed imperviousness. The eight rules called for extended detention storage equivalent to the runoff volume generated from 0.25, 0.50, 0.75, 1.00, 1.25, 1.50, 2.00 and 3.00 inches of rainfall over the specified watershed time of concentration. These rainfall depths were multiplied by the runoff volume coefficient (C'), derived from equation B.8, to obtain a storage volume for each increment of I. The storage volume specified under each sizing rule as a function of I are given below in Table B.1.

Table B.1: Extended Detention Storage Volume For Selected Sizing Rules

SIZING RULE Runoff Volume in inches for rainfall depth of:	VOLUME OF EXTENDED DETENTION STORAGE (inches)										
	<---- degree of watershed imperviousness (percent) --->										
	0	10	20	30	40	50	60	70	80	90	100
0.25 inch	.03	.05	.07	.08	.10	.11	.13	.15	.16	.18	.20
0.50 "	.07	.10	.13	.16	.20	.23	.26	.29	.33	.36	.39
0.75 "	.10	.15	.20	.25	.30	.34	.39	.44	.49	.54	.59
1.00 "	.13	.20	.26	.33	.39	.46	.52	.59	.65	.72	.78
1.25 "	.17	.25	.33	.41	.49	.57	.65	.74	.82	.90	.98
1.50 "	.20	.30	.40	.49	.59	.69	.79	.88	.98	1.08	1.17
2.00 "	.27	.40	.53	.66	.79	.92	1.05	1.18	1.31	1.44	1.57
3.00 "	.40	.60	.79	.99	1.18	1.38	1.57	1.77	1.96	2.15	2.35

Two assumptions had to be made about the nature extended detention storage volume; the storage is considered to be "dead", i.e., the storage is in addition to that provided for control of the design storm, and 2) the storage volume has detention time well in excess of the time to peak of the post-development storm. (e.g, 24 to 40 hours of detention as compared to a time to peak of an hour or less: see Figure B.1)

Step 5: Subtract Storage From Post-Development Runoff Volumes

The net effect of extended detention storage is to store a fraction of the incoming runoff volume and release it later so that it does not materially influence the uncontrolled post-development hydrograph. In the analysis here, the peak discharge occurs within 10 to 80 minutes after the storm, whereas extended detention volumes are stored and released over an interval of 24 hours or more. Thus, it is possible to subtract out the extended detention storage volume (in inches) from the total storm runoff volume (in inches) for an event to determine if the storage is sufficient to prevent a pre-development bankfull flood. Thus, for each recorded rainfall event;

(EQ B.10): $Xvol = Qvp - Xst$

where Xvol= Uncontrolled post development runoff volume (in)
Qvp = Initial post-development runoff volume (in)
Xst = Extended detention storage volume (in)

The next step involved applying equation B.10 to the time series of uncontrolled bankfull floods identified in Step 3 for each of the eight storage rules. Three operating rules were used to evaluate the Xvol for each bankfull flooding event.

1. If Xvol was negative for a particular storm, this meant that the entire storm runoff volume was subject to extended detention, and no flood (NF) occurred.

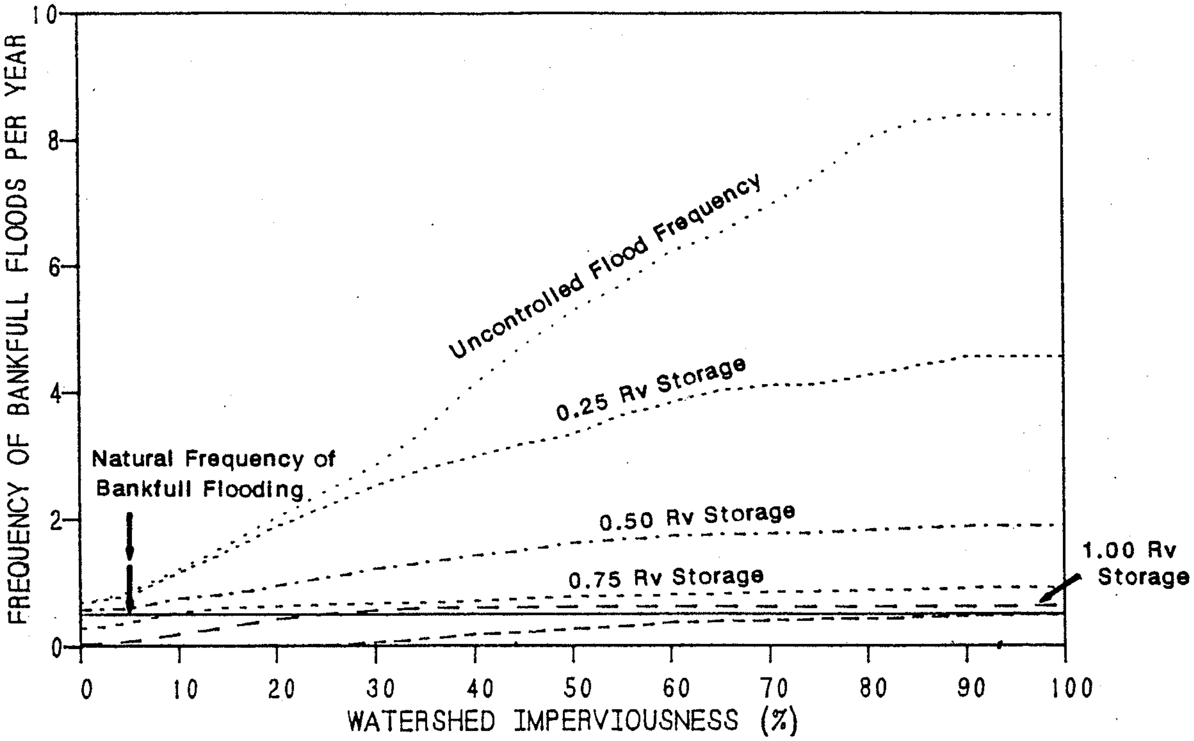
2. If Xvol was positive, but was less than 60% of the pre-development bankfull runoff volume (Qvp), the storm was considered a small flood (SF). For storm events in this range, enough extended detention storage (Xst) was provided so that most of the post-development runoff volume (Qvd) was delayed until well after the original pre-development bankfull flood peak (Qp). The effect of the storage is to reduce the the peak discharge below the pre-development bankfull flood (Qp). The 60% rule is used as an approximation of the typical storage volume needed to control the predevelopment bankfull flood, based on experience with storage routing models (Harrington, personal communication). Ideally, storage routing should be performed on each hydrograph in the bankfull flooding time series to get a more accurate measure of the effect of Xst on Qp. However, the slightly less accurate 60% cut-off rule was used to reduce the number of computations needed in the program.

3. If Xvol was positive, and greater than 60% of Qvp, then the storm event still produced a bankfull flood (BF), and erosive conditions were presumed to still occur downstream.

Step 6: Compute New Frequency of Bankfull Floods After Storage

The number of bankfull floods (BF) recorded after extended detention storage was provided (Operating Rule 3, above) was then tabulated in a three-way matrix (for each increment of watershed time of concentration, watershed imperviousness, and extended detention storage). These values in the matrix were then compared to the original, uncontrolled bankfull flooding frequency curve for all storm events (derived in Step 3), to examine the change in bankfull flooding frequency which could be attributed to extended detention storage. The results, again averaged over all watershed times of concentration is shown in Figure B.4, below.

Figure B.4: Effect of Extended Detention on Post-development Bankfull Flooding Frequency



NOTE: Extended detention storage (STOR) equals the volume equivalent to the given rainfall depth times the runoff coefficient.

Discussion

The curves shown in Figure B.4 suggest that extended detention storage equivalent to the runoff produced by a 0.75 to 1.00 inch storm should be capable of reproducing the natural, pre-development frequency of the bankfull floods. However, a slightly more generous storage rule should be used for design purposes (1.0 to 1.5 storm*C') to account for some of the conservative assumptions used in the analysis. Specifically, two assumptions were made that probably underestimate the frequency of post-development bankfull flooding, and consequently, overestimate the effectiveness of extended detention storage in reducing the frequency. They are:

1. The assumption that watershed time of concentration (Tc) does not change as a result of development. Clearly, Tc will be reduced after drainage

patterns and cover are changed in the watershed. From EQ B.1, it is also evident that this effect should lead to an increase in the frequency of of post-development bankfull flooding. However, because change in Tc as a result of development is unique to each site, it is not possible to derive a systematic relationship relating the change in Tc to I (or any other measure of watershed development).

2. The assumption that the rainfall record used in the analysis includes all storm events that could produce a bankfull flood, for all increments of I. The NWS record only recorded the monthly maximum rainfall intensity for each time of concentration. It is entirely possible, and probably quite likely, that more than one storm event a month is intense enough to produce a bankfull flood, particularly during the summer months when thunderstorms are common. Ideally, the entire analysis should be performed on a continuous and longer record of rainfall intensity to get the most accurate frequency curve.

In conclusion, the analysis presented here is a preliminary attempt to provide a quantitative framework for examining bankfull flooding frequency in small streams. The results should be viewed with some caution, as the method has not yet been tested with actual runoff data for streams.

GLOSSARY

aggregate

Term for the stone or rock gravel needed to fill in an infiltration BMP, such as a trench or porous pavement.

anti-seep collar

A plate, attached to the barrel running through an embankment of a pond, that prevents seepage of water around the pipe.

Austin Triangle

Sediment control device consisting of a long triangular pipe frame, enclosed in heavy gage fencing material, and wrapped in filter fabric.

background load

Naturally occurring levels of pollutants in a stream prior to watershed development.

bankfull discharge

A flow condition where streamflow completely fills the stream channel up to the top of the bank. In undisturbed watersheds, the discharge condition occurs on average every 1.5 to 2 years and controls the shape and form of natural channels.

barrel

The concrete or corrugated metal pipe that passes runoff from the riser through the embankment, and finally discharges to the pond's outfall.

baseflow

The portion of stream flow that is not due to storm runoff, and is supported by groundwater seepage into a channel.

bedload

The sediment in a stream channel that mainly moves by jumping, sliding or rolling on or very near the bottom.

benthic organisms

Organisms living in or on bottom substrates in aquatic habitats.

berm, earthen

An earthen mound used to direct the flow of runoff around or through a BMP.

best management practice (BMP)

Structural devices that temporarily store or treat urban stormwater runoff to reduce flooding, remove pollutants, and provide other amenities.

bioassay

Laboratory tests used to determine the response of organisms to specified conditions relating to the natural environment (e.g., water quality).

biochemical oxygen demand (BOD)

The quantity of oxygen consumed during the biochemical oxidation of matter over a specified period of time (see also COD).

borings

Cylindrical samples of a soil profile used to determine infiltration capacity.

channel erosion

The widening, deepening, and headward cutting of small channels and waterways, due to erosion caused by moderate to large floods.

check dam

(a) A log or gabion structure placed perpendicular to a stream to enhance aquatic habitat. (b) An earthen or log structure, used in grass swales to reduce water velocities, promote sediment deposition, and enhance infiltration.

chemical oxygen demand (COD)

A monitoring test that measures all the oxidizable matter found in a runoff sample, a portion of which could deplete dissolved oxygen in receiving waters.

clay lens

A naturally occurring, localized area of clay that acts as an impermeable layer to runoff infiltration.

dead storage

The portion of a pond or infiltration BMP which is below the elevation of the lowest outlet of the structure.

denitrification

A biological process in which nitrate (NO3), a compound of nitrogen often found in sewage or water, is turned into nitrogen gas, which can dissipate into the atmosphere.

design storm

A rainfall event of specified size and return frequency (e.g., a storm that occurs only once every 2 years) that is used to calculate the runoff volume and peak discharge rate to a BMP.

detention

The temporary storage of storm runoff in a BMP, which is used to control the peak discharge rates, and which provides gravity settling of pollutants.

detention time

The amount of time a parcel of water actually is present in a BMP. Theoretical detention time for a runoff event is the average time parcels of water reside in the basin over the period of release from the BMP.

downstream seepage

The horizontal movement of runoff through the soil layer, which may cause damage to nearby building foundations.

drawdown

The gradual reduction in water level in a pond BMP due to the combined effect of infiltration and evaporation.

dryfall

The deposition of atmospheric pollutants on the land surface.

emergent plants

Aquatic plants that are rooted in the sediment but whose leaves are at or above the water surface. These wetland plants often have high habitat value for wildlife and waterfowl, and can aid in pollutant uptake.

eutrophication

The process of over-enrichment of water bodies by nutrients often typified by the presence of algal blooms.

event mean concentration (EMC)

The average concentration of an urban pollutant measured during a storm runoff event. The EMC is calculated by flow-weighting each pollutant sample measured during a storm event.

exfiltration

The downward movement of runoff through the bottom of an infiltration BMP into the soil layer.

fecal coliform bacteria

Minute living organisms associated with human or animal feces that are used as an indirect indicator of the presence of other disease causing bacteria.

filter fabric

Textile of relatively small mesh or pore size that is used to (a) allow water to pass through while keeping sediment out (permeable), or (b) prevent both runoff and sediment from passing through (impermeable).

first flush

The delivery of a disproportionately large load of pollutants during the early part of storms due to the rapid runoff of accumulated pollutants. The first flush of runoff has been defined several ways (e.g., one-half inch per impervious acre).

flood frequency

The frequency with which the maximum flood may be expected to occur at a site in any average interval of years. Frequency analysis defines the "n-year flood" as being the flood that will, over a long period of time, be equaled or exceeded on the average once every "n" years.

flood plain

For a given flood event, that area of land adjoining a continuous watercourse which has been covered temporarily by water.

flow-weighting

A statistical technique used to adjust a series of pollutant concentration measurements for the effect of flow.

forebay

An extra storage area provided near an inlet of a BMP to trap incoming sediments before they accumulate in a pond BMP.

freeboard

The space from the top of an embankment to the highest water elevation expected for the largest design storm stored. The space is required as a safety margin in a pond or basin.

frost-heave

The upward movement of soil surface due to the expansion of ice stored between particles in the first few feet of the soil profile. May cause surface fracturing of asphalt or concrete.

gabion

A large rectangular box of heavy gage wire mesh which holds large cobbles and boulders. Used in streams and ponds to change flow patterns, stabilize banks, or prevent erosion.

hardness

A measure of the concentration of dissolved calcium carbonate in water. Hardwater has high levels, and causes scaling in pipes that increases frictional resistance to flow.

headwater stream

A stream forming the source of another and larger stream.

hydrograph

A graph showing variation in the water depth or discharge in a stream or channel, over time, at a specified point of interest.

impervious area

Impermeable surfaces, such as pavement or rooftops, which prevent the infiltration of water into the soil.

infiltration

(a) The downward movement of water from the surface to the subsoil. (b) The infiltration capacity is expressed in terms of inches/hour.

invert elevation

The vertical elevation of a pipe or orifice in a pond which defines the water level.

level-spreader

A device used to spread out stormwater runoff uniformly over the ground surface as sheet flow (i.e., not through channels). The purpose of level spreaders are to prevent concentrated, erosive flows from occurring, and to enhance infiltration.

low flow channel

An incised or paved channel from inlet to outlet in a dry basin which is designed to carry low runoff flows and/or baseflow, directly to the outlet without detention.

overflow rate

Detention basin release rate divided by the surface area of the basin. It can be thought of as an average flow rate through the basin.

peak discharge
The maximum instantaneous rate of flow during a storm, usually in reference to a specific design storm event.

peak-shaving
Controlling post-development peak discharge rates to pre-development levels by providing temporary detention in a BMP.

pilot channel
A riprap or paved channel that routes runoff through a BMP to prevent erosion of the surface.

plug flow
A flow value used to describe a constant hydrologic condition. Often used in the context of describing a plug flow model, or a model that is not applied with time variable flow conditions.

Rational formula
A simple technique, developed in the 1900's, for estimating peak discharge rates for very small developments, based on the rainfall intensity, watershed time of concentration, and a runoff coefficient.

release rate
The rate of discharge in volume per unit time from a detention facility.

retention
The holding of runoff in a basin without release except by means of evaporation, infiltration, or emergency bypass.

retrofit
To install a new BMP or improve an existing BMP in a previously developed area.

return interval
A statistical term for the average time of expected interval that an event of some kind will equal or exceed given conditions (e.g., a stormwater flow that occurs every 2 years).

riparian
A relatively narrow strip of land that borders a stream or river, often coincides with the maximum water surface elevation of the 100 year storm.

riprap
A combination of large stone, cobbles and boulders used to line channels, stabilize banks, reduce runoff velocities, or filter out sediment.

riser
A vertical pipe extending from the bottom of a pond BMP that is used to control the discharge rate from a BMP for a specified design storm.

senescence
The annual die-back of aquatic plants at the end of the growing season.

sheetflow
Runoff which flows over the ground surface as a thin, even layer, not concentrated in a channel.

short circuiting

The passage of runoff through a BMP in less than the theoretical or design treatment time.

softwater

Water with low concentrations of calcium (CaCO3) ions.

soil group, hydrologic

A classification of soils by the Soil Conservation Service into four runoff potential groups. The groups range from A soils, which are very permeable and produce little runoff, to D soils, which are not very permeable and produce much more runoff.

soil strata

The various horizontal layers of sedimentary rock (soil).

sorption

The physical or chemical binding of pollutants to sediment or organic particles.

spillway

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

Stokes Law Type I Sedimentation

A model for the settling characteristics of particulate materials in bodies of water; whereby, coarser materials are deposited first, followed by finer-sized fractions.

stormflow

The portion of flow which reaches the stream shortly after a storm event.

storm pulse

A high concentration of urban pollutants found in a stream for a short period of time, following a rainstorm.

streamflow

Water flowing in a natural channel, above ground.

subgrade

A layer of stone or soil used as the underlying base for a BMP.

substrate

The natural soil base underlying a BMP.

swale

A natural depression or wide shallow ditch used to temporarily store, route, or filter runoff.

test well

A device installed in an infiltration BMP to monitor infiltration rates.

time of concentration

The time required for surface runoff from the most remote part of a drainage basin to reach the basin outlet.

TR-20

A watershed hydrology model developed by the Soil Conservation Service act that is used to route a design storm hydrograph through a pond.

underdrain

Plastic pipes with holes drilled through the top, installed on the bottom of an infiltration BMP, or sand filter, which are used to collect and remove excess runoff.

water quality BMP

A BMP specifically designed for pollutant removal.

water table

The upper surface or top of the saturated portion of the soil or bedrock layer, indicates the uppermost extent of groundwater.

wetfall

The deposition of atmospheric pollutants on the land surface that are washed out by precipitation.

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