GREEN TECHNOLOGY: THE DELAWARE URBAN RUNOFF MANAGEMENT APPROACH

A TECHNICAL MANUAL FOR DESIGNING NONSTRUCTURAL BMPs TO MINIMIZE STORMWATER IMPACTS FROM LAND DEVELOPMENT

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CHAPTER 1 GREEN TECHNOLOGY OVERVIEW

1.1 BACKGROUND

By early 1970, the deleterious effects of urban runoff on stream water quality had become apparent (Coughlin and Hammer, 1973, and sources cited therein). At the time though, relatively few studies had focused on the nature, extent, and effects of urban runoff. Following passage of the Clean Water Act in 1972, Section 208 Reports from the states began to accumulate a considerable body of information. By the late 1970s, these reports had indicated that urban runoff is a significant source of Nonpoint Source (NPS) Pollution. However, it was difficult to determine the particular effects of urban runoff on water quality due to interferences from other pollutant sources (USEPA, 1984). To address the issue more thoroughly, the Nationwide Urban Runoff Program (NURP) in the early 1980s monitored urban runoff from 28 sites with a wide variety of land uses (USEPA, 1984).

Urban runoff has been known as a source of pollution of water bodies (Hartigan and others, 1979; USEPA, 1984). Urban and suburban regions contribute higher (NPP) loadings on a per acre basis than rural watersheds (CPB 1990). While urban runoff itself may have significant levels of pollutants, the way it travels to receiving waters from developed areas may have even greater implications. After traveling over impervious surfaces coated with fine sediments and their associated nutrients, metals and hydrocarbons, most runoff enters streams directly through piped conveyance systems. These connected impervious areas are referred to as Effective Impervious Areas (EIA). Pollutant loads from EIA surfaces are much higher than those from isolated impervious areas filtered by overland flow at a much slower rate. Instead of percolating into the soil, precipitation from EIA surfaces is diverted to runoff that quickly flows into the aquatic ecosystem (WDE, 1992).

Urban runoff impairs stream systems through two major processes: changes in the water chemistry and changes in stream hydrology. Studies show that urban stream pollution may substantially impair the viability of benthic macroinvertebrates. Urban runoff also fundamentally affects hydrology by increasing storm runoff while reducing recharge, and thus lowering base flow. As a result, stream banks erode on a regular basis, filling the streams with sediment and smothering organisms. The organisms remaining are further stressed by runoff pollutants, as well as by the low flows and elevated temperatures resulting from reduced recharge. Under these circumstances, runoff pollutant concentrations and available dissolved oxygen often approach toxic levels. As a result of these processes, streams in urbanized watersheds have been greatly impaired due to runoff from upland impervious areas. Changes in hydrology seem to be the more pernicious, and harder to rectify.

The issue of stream bank erosion is particularly important. A growing body of research indicates that increased frequency of bankfull flooding due to development is the dominant process that impairs urban streams. In areas where streambanks comprise erosive alluvial soils typical of floodplains, the increase in the frequency and duration of bankfull flow from urban runoff rapidly degrades stream banks. Instream bank erosion may be a dominant source of total suspended solids (TSS) in urban streams.

The threshold at which streams begin to show signs of impairment is surprisingly low, beginning at roughly 5% EIA, typical of low-density residential development. In areas of higher density development, streams may be heavily impaired, with the native community almost entirely replaced by more opportunistic species. The fish community thus shifts from game fish to rough fish. In heavily urbanized streams, worms may be the only organisms surviving.

The lakes and inland bays into which streams and rivers discharge are also adversely affected by urban runoff. In these slow moving or still waters exposed to full sun, excess nutrients cause algae to proliferate inordinately, resulting in eutrophication. In freshwater streams, ponds, and upper tidal rivers, excess phosphorus is the most damaging nutrient. In the lower tidal rivers and inland bays, excess nitrogen is the most damaging nutrient.

Recent research indicates that the current paradigm in stormwater management regulations requiring structural Best Management Practices (BMPs) such as wet ponds does not do enough to alleviate this impairment and, in certain cases, may actually exacerbate aspects of urban runoff. In response, the Green Technology approach has been formulated to mitigate the effects of development on stream ecosystems through the use of innovative nonstructural BMPs. The goal is not only to maintain the health of existing streams, but also to build a foundation for efforts to restore impaired streams.

This Technical Manual reviews current understanding of the impacts of urban runoff upon stream ecosystems and their associated riparian zones. This manual then presents how Green Technology BMPs can effectively address these impacts, and then how to properly design the most appropriate BMP. The technical approach set forth in this Manual is supported by a spreadsheet model and Users Manual in Appendix A to assist in the design of Green Technology BMPs. BMP Design standards, specifications and details are presented in Appendix B.

1.2 THE GREEN TECHNOLOGY BMP APPROACH

In the Green Technology approach, centralized treatment and/or storage facilities located at the "end of pipe" discharge from developed sites are classified as structural BMPs. While structural BMPs such as stormwater ponds and wetlands can be effective in controlling peak flows from the site, current regulatory requirements for these structures do not address the frequent storms that erode stream banks, and do little or nothing to promote recharge. Furthermore, structural BMPs can contribute to downstream flooding when discharges from separate on-site structural BMPs overlap. Structural BMPs can be effective in pollutant removal; but since they generally omit recharge, consume space, and require extensive maintenance, they are less appropriate for the task. There is an emerging recognition that wet detention structural BMPs contribute to elevated stream temperatures, and discharge algae laden effluent, which can substantially degrade the benthic community in the receiving stream.

As a result, many progressive agencies are promoting the Green Technology approach, which is designed to intercept runoff from rooftops, parking lots and roads as close as possible to its source, and direct it into vegetative recharge/filtration facilities incorporated into the overall site design and runoff conveyance system. Green Technology BMPs defined in this Manual include conservation site design, impervious area disconnection, conveyance of runoff through swales and biofiltration swales, filtration through filter strips, terraces, bioretention facilities, and recharge through infiltration facilities. These BMPs form the basis of Green Technology at the site engineering level.

As vegetated structures that do not rely on detention, these BMPs are “Green”. However, while Green Technology BMPs may seem less complex than structural detention BMPs, procedures for their proper design require the same hydrologic and hydraulic methods used in designing structural BMPs. The Green Technology approach also includes a quantitative approach for estimating pollutant loads, and projecting how well a particular design will remove such pollutants. Hence it is a “Technology”, capable of providing realistic estimates of pollutant loading and removal, while also addressing hydrologic and hydraulic parameters involved in urban site design. The detailed design principles set forth in this Manual are incorporated into a spreadsheet program, the “Delaware Urban Runoff Management Model”, or DURMM.

1.3 GREEN TECHNOLOGY BMPs

The BMPs addressed in Green Technology and pertinent aspects of their design and performance are briefly summarized below:

> Conservation Site Design - Site design standards to reduce the extent of impervious surfaces and increase the extent of wooded areas are a key element of this approach, as expressed in the “Conservation Design Manual for Stormwater Management” by DNREC and the Brandywine Conservancy (1997). This Conservation Design Manual addresses many of the background issues in urban runoff and discusses conservation design methods in detail. The reader is referred to it for greater details in Conservation Design principles. Green Technology provides a quantitative approach to define the benefits of Conservation Design.
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> Source Area Disconnection – Disconnection is the process of directing runoff from impervious surfaces over adjacent vegetated surfaces, providing infiltration and pollutant removal. Green Technology quantifies the runoff reductions by disconnecting flow from impervious surfaces as it discharges onto adjacent pervious areas.
>
> Filter Strips – Filter strips spread runoff uniformly over a filtering surface of vegetation, providing infiltration and pollutant removal. Filter strips can provide substantial treatment if not overloaded by sediment and runoff. Green Technology quantifies the runoff reductions and pollutant removal of filter strips.
>
> Biofiltration Swales/Grassed Swales - Biofiltration swales convey runoff at shallow flow depths through wide swales. They can be very effective in removing Total Suspended Solids (TSS) and adsorbed metals, although less effective in terms of nutrients. While swales are not thought to be capable of quantity management, designs incorporating check dams can provide substantial attenuation of peak flows. Green Technology quantifies the runoff reductions and pollutant removal of overland conveyance through properly designed swales.

Terraces - Terraces are swales extending across gentle slopes designed to intercept runoff and increase the potential for infiltration. In terms of pollutant removal, terraces operate as filter strips, as runoff flows into them from upslope. They are similar to swales in terms of runoff response. Green Technology quantifies the runoff reductions and pollutant removal of overland conveyance through properly designed terraces.

Bioretention Structures - These are landscaped pocket depressions designed to infiltrate runoff through an engineered soil media. Incorporated into the urban landscape, they can provide substantial filtering and nutrient transformations before runoff is discharged into the conveyance system. Ongoing research suggests that this BMP can be designed to provide substantial soluble phosphorus removal capabilities, unlike most other BMPs. Green Technology quantifies the runoff reductions and pollutant removal of overland conveyance through properly designed bioretention structures.

Infiltration Practices - Most Green Technology BMPs incorporate infiltration as part of the treatment process. Specific infiltration facilities include infiltration trenches. Infiltration trenches located in swales provide additional wetted surface area and storage volume, and often they can be designed to penetrate shallow impermeable soil profiles to recharge deeper soil horizons. Green Technology quantifies the runoff reductions of infiltration trenches.

Complementing these engineered BMPs, Riparian Buffer Systems (RBS) and Stream Bank Restoration (SBR) BMPs are other important Green Technology systems that can enhance receiving waters. These BMPs provide substantial improvements in stream habitat and stability, as well as reducing pollutants from urban runoff. RBSs provide considerable benefits to streams through shading, bank stabilization and litterfall. RBSs can also provide substantial runoff filtering and pollutant removal when conditions are favorable. Since RBSs are sensitive to concentrated flows, design procedures to ensure sheet flow through level spreaders, filter strips and parallel swales can be incorporated into the design of this BMP. A companion document specifically focused on RBS design is being prepared by DNREC.

Stream Bank Restoration differs from other BMPs in that it provides no direct hydrological controls, nor does it remove pollutants from upland runoff. However, by stabilizing eroding stream banks, it may be the most effective mitigation measure for unstable streams stressed by urban runoff. Technical approaches for design of SBR BMPs are not included in this Manual. Designers should review the available literature about SBR design.

1.4 MANUAL SUMMARY

Unfortunately, while there is great interest in using Green Technology BMPs, there are remarkably few rigorous procedures available for the engineering and regulatory community to utilize in designing them and evaluating their effectiveness. Many regulatory programs use a straightforward runoff volume approach, in which the increase in small storm runoff volume due to land development is to be treated and/or retained on site. However, this approach typically assumes a constant runoff volume in proportion to rainfall amount, and does not route runoff through nonstructural BMPs. Instead, simplified volume/outflow equations are extracted from the literature, without addressing the processes involved during storm events. Where this approach leads to over-design, it may be beneficial if the original reduction targets are inadequate, otherwise it causes unnecessary expense. Where it leads to under-design, the hydrological impacts are not adequately mitigated.

DNREC has created DURMM to provide a more rigorous hydrological design tool for Green Technology BMPs. A spreadsheet program is provided that incorporates modified TR-20 storm hydrology to project the hydrological response from contributing source areas. It segregates directly connected runoff from that which flows overland. It provides routines that account for the reductions in peak flow due to overland conveyance. It also includes simplified estimates of the storage volume required for detention in different nonstructural BMPs.

The process of BMP design involves a spreadsheet file for each source subarea and its array of BMPs. Discrete combinations of hydrological soil group and land cover are averaged to generate composite Curve Numbers (*CN*) for the pervious and impervious portions of each source area. Natural areas are treated separately. Impervious areas are also calculated separately, and routed according to the extent of their linkage with adjacent pervious surfaces. The resulting runoff parameters from the source area worksheet are imported into the BMP hydraulic design worksheet. The worksheet routes the source area runoff volumes and peak flows through the BMP based upon the input parameters.

Pollutant loading is calculated by applying typical event mean concentrations (EMCs) to the runoff volume allocated to each type of pervious and impervious surface. By segregating subarea loads according to the type and extent of land cover, the discrete source area approach used in the hydrologic calculations refines accuracy in estimating total pollutant loads. Pollutant removal by the BMP is based upon physical parameters such as slope, pretreatment volume, hydraulic residence time, surface/volume ratio, filter media type, and underlying infiltration characteristics. Given these factors, pollutant load reduction is calculated by algorithms relating input concentrations and decay transformations to estimate mass removal for each pollutant of concern.

The reported pollutant removal effectiveness of BMPs can be highly variable. However, by incorporating hydrologic and hydraulic parameters in runoff routing, and addressing the various removal processes as discrete algorithms within a BMP, better estimates of removal rates are possible. Some variability in projected removal rates is acceptable in any event, since hydrological changes are recognized as perhaps the primary impact of runoff. Furthermore, polluted runoff from the most frequent storms that causes the greatest stress can often be eliminated by the infiltration components of nonstructural BMPs.

DURMM not only provides the tools necessary for designing Green Technology BMPs, typical details of these BMPs in AutoCAD™ format are provided for use by the engineering community. Procedures for site analysis are provided, particularly as they relate to disconnection of impervious runoff. Given a thorough site analysis, locations where BMPs are most needed become apparent. As a Windows™ interface, data from AutoCAD™ or similar design programs can easily be imported into DURMM during project design. The particulars involved in the design of each type of BMP are readily accessible during this process so that calculation of source area impacts and BMP performance becomes an integrated procedure, and the BMP is designed as a fundamental part of the entire project design process.

To properly address the issue of urban runoff and best mitigate its impacts through Green Technology BMPs, it is first necessary to thoroughly examine the underlying issues. Chapter 2 summarizes the literature on urban runoff impacts, Chapter 3 summarizes the literature on pollutant loads, and Chapter 4 addresses urban runoff criteria. Details of the hydrology of urban runoff are set forth in Chapter 5, and runoff hydraulics and DURMM routing are discussed in Chapter 6. Chapter 7 summarizes pollutant removal processes, while the Green Technology BMPs are described in Chapters 8 though 11.

CHAPTER 2 URBAN RUNOFF IMPACTS

2.1 INTRODUCTION

There are considerable numbers of studies that relate the presence of uncontrolled urban runoff to impairment of streams throughout the US. Studies have demonstrated impacts upon the habitat and native stream ecosystems in the Puget Sound area (WDE, 1992; Horner and others, 1996; Booth and Jackson, 1997), the Midwest (Richards and Host, 1994; Dreher, 1997), and the mid-Atlantic (Coughlin and Hammer, 1973; Schueler and Claytor, 1996; Kennen, 1999; Maxted and Shaver, 1997, Jones and others, 1996).

Urban runoff can affect streams through two processes: its pollutants either stress or totally alter the native benthic community, thus eliminating game fish; and its modification of stream hydrology can result in substantial loss of habitat. Urban runoff can also affect lakes and estuaries by increasing nutrient loads, leading to eutrophication. Consequences include red tides, loss of fisheries, and waters too foul for recreation. USEPA (1999) provides an excellent review of the impacts of urban runoff on receiving streams. The following sections address these processes in detail to better define the problem, its causes, and how nonstructural BMPs can be better designed to mitigate impacts of urban runoff.

2.2 IMPACTS OF URBAN RUNOFF TOXICITY ON BENTHIC COMMUNITIES

Toxic compounds in urban runoff can substantially impair the viability of benthic macroinvertebrates. While these bottom-dwelling insects, worms, and crustaceans may not be directly important to human uses (except fly fishermen), they are a robust indicator of overall stream health. Where a healthy benthic community has shifted to a few pollution-tolerant taxa, a diverse fish assemblage including game fish disappears, to be replaced by a few species of rough fish, if any. Therefore, the Index of Biotic Integrity (IBI) used to evaluate fish assemblages are very closely related to indices of benthic community health. As it is much easier to obtain quantitative information on benthic communities, methods to determine benthic indices such as the USEPA Rapid Bioassessment Protocol (RBP) are almost universally used by the states to evaluate the extent of stream impairment in their 305 (b) Reports to Congress.

Since water chemistry is inextricably linked with the hydrology of urban runoff, it is difficult to segregate the relative influence of hydrological and chemical impacts. However, several studies have explored the impacts of exposure to urban runoff using methods that minimize hydrologic variables. While no study has conclusively demonstrated the toxicity of urban runoff to sensitive benthic taxa, several studies examine the effects of urban runoff toxicity upon test organisms, using the Whole Effluent Toxicity (WET) protocols requiring a 48 hour exposure for acute tests, and a longer exposure for chronic tests.

Urban runoff contains a wide variety of toxic compounds and metals. From a commercial site in the Lincoln Creek watershed, a heavily urbanized stream in Wisconsin, lead EMCs exceeded USEPA's acute toxicity standard in 90% of runoff events (WDNR, 1989, as cited in Bannerman, 2000). In the Dallas area, zinc was found in 100% of all sites, with median concentration in residential sites of 65 µg/l, and 130 µg/l in commercial sites. The acute water quality criterion of 112 µg/l was exceeded in 36% of the samples (Waller and others, 1997).

In California, concentrations of copper, lead, cadmium and zinc in urban runoff sediments are from 10 to 50 times the background levels found in sediments originating from open areas. Copper exceeded the acute objectives in 70% of observations, and zinc 61% of the time. However, soluble copper, which is considered the toxic species, exceeded the chronic objective only 5% of the time (Cooke and others, 1997). Chronic exposures to runoff from a heavy industrial site in California were lethal to *C. dubia* in 100% of observations, while runoff from residential and commercial sites was lethal 50% of the time, and moderately to highly toxic for the balance. Toxicity in the receiving Coyote Creek varied from event to event, ranging from no observed effect in most cases, to high levels of mortality (Cooke and others, 1997).

Accumulation of metals in the sediments has been recognized as potentially hazardous (Hartigan and others, 1979), and repeated resuspension of contaminated sediments during frequent storms may pose the greatest long term toxic impacts to fisheries (USEPA, 1984, Myers and others, 1985; WDE, 1992). For this reason, Livingston and others (1995) consider sediment sampling a better measure of potential urban runoff toxicity than water chemistry. Resuspension of copper from sediments during stormflow below an abandoned factory has been implicated as a cause of significant mortality to benthic organisms (Diamond, 1996). Locally, the Red Clay Creek in the Piedmont is a good example of the adverse effects of industrial discharges upon stream sediments.

Insecticides are another major toxic contaminant in urban runoff. Used extensively in residential settings, excess diazinon in effluent from many STPs in the southern USA has been implicated in their failure to meet discharge limitations. In Wisconsin, diazinon was reported in 20% to 49% of all samples, with a mean EMC of 0.11 µg/l (Bannerman and others, 1996). In the Dallas area, Waller and others (1995) report that diazinon was found in 100% of commercial and residential sites, and 83% of industrial sites. Median concentration in residential sites was 0.55 µg/l, with 20 of 31 observations in excess of 0.35 µg/l. For the water flea *C. dubia*, this is one value reported for the concentration at which 50% die (LC50). The LC50 of diazinon for *C. dubia* has been reported as high as 0.9 µg/l (Fernandez-Casalderry and others, 1994), so fewer events would exceed this LC50. Thus it is not surprising that acute WET tests of urban runoff from these sites showed minimal toxicity to *C. dubia*, even though most exceeded the acute criteria for zinc as well (Waller and others, 1995). In December, 2000, the USEPA passed rules to phase out the use of diazinon, so this insecticide should become less of a problem as its use declines.

The time scale of exposure is a particularly important issue. Herricks and others (1995) note that the exposure period for acute and chronic exposure in WET tests does not account for the time scale involved in streams, where chronic exposure lasts for the lifetime of the organisms. Herricks and Milne (1996) also noted that mortality from an acute exposure was not manifest until a chronic time period had elapsed. Nonetheless, Herricks and others (1997) reported mortality of *C. dubia* from exposure to the first flush of runoff from Lincoln Creek in WET tests. However, field tests of caged rotifers (*H. azteca*) and native isopods (*Asellus* sp.) showed no event-related effects on mortality.

To isolate flow effects, Crunkilton and others (1997) used mesocosms (essentially aquariums) of test organisms filled with circulating stream water. They reported that 93% of the tests showed significant mortality in *C. dubia* after a 14 day exposure to runoff in Lincoln Creek, and 100% of the tests showed significant mortality in the minnow *P. promelas* after a 61 day exposure. Growth rates of *P. promelas* during long exposures were also reduced relative to controls. Much less effect was seen in shorter duration exposures, supporting the results of Herricks and others (1997). They also note that more sensitive criteria such as biomass accumulation may be better suited to examine the effects of runoff toxicity.

Even more noteworthy, there was little difference in mortality between runoff and baseflow inputs to the mesocosms. This suggests that sediments may accumulate toxic pollutants from runoff events, and supposedly “clean” baseflow becomes toxic as it upwells through bottom sediments and absorbs pollutants.

Bioassay exposure to polyaromatic hydrocarbons (PAH) extracted from stormflow runoff from Lincoln Creek also had substantial effects. However, there was a lesser effect from base flow (Villeneuve, 1997, as cited in Bannerman, 2000), suggesting that some PAHs are more mobile and less likely to be sequestered in the sediments. Recent research indicates that the more soluble PAHs such as benzene and naphthalene are generally more toxic, while larger species bound to sediments exert their effects through bioaccumulation in sediment burrowing organisms (Standley, pers. comm.).

These studies show that urban runoff is toxic when levels of pollutants exceed the threshold particular to a test procedure. Some consider *C. dubia* a most sensitive indicator of toxicity (Waller and others, 1995), so WET sensitivity of *C. dubia* may overstate life cycle sensitivity of other benthic organisms. Sensitivity to long term exposure generally occurs at orders of magnitude below lethality at acute exposure. By inhibiting growth, reproduction, and resistance to stress, sublethal levels of toxic pollutants will have substantial effects on benthic communities and will select for pollutant tolerant taxa.

Undoubtedly, these studies strongly implicate urban runoff toxicity as a factor in the impairment of benthic community structure. Since most of the toxic pollutants such as metals are associated with sediments, measures that reduce TSS are the primary mechanism for reducing toxic runoff pollutants. This can be accomplished by either filtration, infiltration or settling BMPs. The design criterion requires that the storage volume be adequate to treat the vast majority of runoff. It must be stressed that infiltration is not the recommended BMP approach where toxic pollutants are mobile and could contaminate groundwater, as in the case of the soluble forms of zinc, copper, soluble PAHs, and many insecticides.

2.3 IMPACTS OF URBAN RUNOFF NUTRIENTS ON LAKES AND INLAND BAYS

Since free flowing shaded streams obtain their energy inputs from the surrounding riparian forest, benthic macroinvertebrate communities in such streams are generally unaffected by elevated levels of the nutrients phosphorus and nitrogen. However, in still water and slower moving reaches exposed to full sun, stream ecosystems are driven by photosynthesis to metabolize nutrients in the water column. In these conditions, typical of tidal streams and inland bays, nutrient loading from urban runoff can be most damaging. Excess nutrients encourage unrestricted growth of algae, leading to eutrophication. As the algae decompose, they consume available oxygen, resulting in nearly complete anoxia at the bottom in many cases. In marshes and lakes, phosphorus is relatively scarce under natural conditions, so excess phosphorus is the most damaging nutrient. In inland bays where nitrogen is limited, excess nitrogen is generally the most damaging nutrient, although bays can be occasionally phosphorus limited.

Although agriculture is widely documented as the most pervasive source of NPS pollution, urban runoff is also implicated in the pollution of water bodies (Hartigan and others, 1979). While the total area may be less than that of rural regions, urban and suburban regions are thought to contribute higher NPS loadings on a per acre basis than rural watersheds (CPB, 1990). This has been attributed to atmospheric deposition of pollutants and nitrate onto impervious surfaces and fertilization of turf (MDE, 1986), as well as the prevalence of onsite septic systems. Nitrate and phosphate concentrations have been found in runoff from shopping centers at levels similar to those from row crops. Originating from pets, fecal coliform counts can be very high in urban runoff (USEPA, 1984).

Septic systems are responsible for well over half the total suburban nitrate loading, and over one third of the suburban phosphorus loading into the inland bays (Ritter, 1986, as cited in Martin 1998). Largely as a result of widespread use of septic systems, suburban residential uses thus account for the second largest source of excess nitrogen to the bays. In the mid-Atlantic Coastal Plain, groundwater levels of inorganic nitrogen (mostly nitrate) in areas developed with septic systems were found to be identical to those under agricultural fields (Reay and others, 1996). However, even in the relatively developed Indian River Bay, it is recognized that agricultural sources of nitrogen are over three times those from urban land uses (Horsley and Witten, 1998).

These effects of excess nutrient loading obviously impair the entire ecosystems in lakes and inland bays. When hypertrophic conditions occur due to excess nutrients, inland bays become subject to increased frequency of Pfisteria outbreaks, resulting in widespread fish kills and health impacts upon people in contact with infested waters. Beyond their effect upon ecosystems health, these effects have considerable economic implications to fisheries and tourism.

Delaware’s inland bays are considered to be the most highly eutrophic estuaries in the Chesapeake Bay region. Previous oyster, soft clam and bay scallop fisheries are now essentially extinct due to habitat loss and poor water quality (CCMP, 1995). Water quality indices and benthic community structure reflect significant impairment, especially in those areas with the least amount of tidal flushing. Dead-end canals constructed for urban areas are particularly impaired, with nearly complete loss of the natural ecosystem. Chemical pollutants in these canals exceed published guidelines in 91% of the sampled areas, and dissolved oxygen levels were below the 5 ppm threshold for aquatic life in 57% of the sampled area (Chaillou and others, 1996).

Urban land uses in the Indian River and Rehoboth bays has doubled from 1986 to 1992. Most of these new urban areas represent conversion of previously forested lands, while the extent of agricultural land uses has remained largely unchanged (Martin and others, 1998). Although agricultural land uses are the primary source of excess nutrient deliveries to the bays, this pattern of land use change suggests that nutrient loading will increase due to development, instead of replacing one source of nutrients with another. Therefore, measures to reduce the amount of nutrients in urban runoff are necessary to avoid further deterioration of Delaware's lakes and inland bays as forested lands are developed.

Since much of the phosphorus in urban runoff is associated with suspended sediments, measures to reduce TSS are the primary mechanism for removal of sediments and particulate phosphorus, using filtration, infiltration, or settling BMPs. On the other hand, much of the total nitrogen (TN) occurs in soluble forms. Likewise, most of the bioavailable phosphate occurs in form of soluble orthophosphate. Therefore, nonstructural BMPS using sedimentation and overland filtration processes do not reduce nitrate or orthophosphate levels substantially. Infiltration BMPs transfer nitrate into groundwater, where it is then eventually discharged as base flow into streams. To varying degrees, wetland, riparian forest buffer and denitrifiyng bioretention BMPs provide mechanisms for removal of nitrate from urban runoff.

2.4 IMPACTS OF URBAN RUNOFF HYDROLOGY ON STREAMBANK STABILITY

Although the toxicity data is suggestive, there do not seem to be any studies that isolate specific toxic effects of urban runoff upon benthic macroinvertebrates from its hydrologic impacts (eg., see Diamond, 1996). While there is ample data from Red Clay Creek implicating PCBs from an industrial discharger, or copper from an abandoned factory in Virginia (Diamond, 1996), these situations are not typical for urban runoff. In the mid-Atlantic region, benthic communities in urbanized watersheds were substantially impaired where no changes in water chemistry were noted, in comparison to those in forested watersheds (Jones and others, 1997). In Ohio, biological impairment was noted in 50% of the stream segments, even though no water quality criteria exceedances were observed (Zucker and White, 1996). Though there may be little doubt about the toxicity of urban runoff to test organisms, Horner and others, (1996) note that changes in stream hydrology and geomorphology resulting from urban runoff are even more pernicious for streams and their benthic communities than runoff toxicity and nutrient loading.

Changes in the character of urban streams in response to urban runoff have been observed for some time. In a landmark study, Hammer (1973) noted that channel cross-sectional areas of urban streams in southeastern Pennsylvania were enlarged by a factor of 10 to 20 times that of rural streams with similar drainage areas. Klein (1979) noted that degradation of Piedmont stream channels in Maryland was correlated with the extent of watershed imperviousness. Krug and Goddard (1986) noted a substantial increase in channel size and sediment delivery in a Midwestern watershed undergoing urbanization. Whipple and others (1991) noted that the extent of erosion of urban streams in Maryland was correlated with the extent of Total Impervious Area (TIA) in the watershed.

Under natural conditions, the flow event that moves the most sediment (known as the maximum of the effective work curve) occurs at an interval from one to two years (Leopold and others, 1964). This recurrence interval corresponds to the natural channel forming event frequency (Wolman and Miller, 1960). The primary impact of urban runoff hydrology is the frequency of bankfull flooding increasing from once every two years or so under natural conditions (Wolman and Miller, 1960) to many times per year after urbanization (Arnold and others, 1980; Booth and Jackson, 1997; Moscrip and Montgomery, 1997).

In Washington, DC, dense urbanization has caused bankfull flooding to occur 10 to 20 times more often than the pre-development frequency (Dunne and Leopold, 1978). Conversion of a watershed in western Washington from forested to medium density residential use (EIA at 29%) is projected to increase the frequency of the 5 year bankfull flood to nearly six times per year (Booth, 1990), nearly a thirty-fold increase in the frequency. This is far greater than the two to six-fold increases reported in earlier studies (Coughlin and Hammer, 1973; Schueler, 1987).

This increase in frequent floods shifts the maximum of the effective work curve to mid-bankfull flow events occurring at least several times per year (MacRae and Rowney, 1992). The increase in frequency of flows above the midbank not only causes erosion of bed material, resulting in channel incision (Harvey and Watson, 1986; Shields and others, 1994); it also over-steepens the banks, so bankfull flooding causes stream banks to erode into the incised channel (Arnold and others, 1980; MacRae, 1991). With roots exposed by bank erosion, remaining riparian trees are more subject to windthrow, further widening the banks (Schueler, 1987). Debris dams are left suspended above the channel, reducing roughness so that even greater downcutting occurs (Booth, 1990). This process is aggravated by the increased magnitude and duration of flows that exceed the critical threshold of non-cohesive materials at the toe of the streambank (MacRae, 1991). As urbanization proceeds, the frequency of these sub-bankfull events exceeding this threshold increases by a factor of roughly three (MacRae and Rowney, 1992), or up to ten (MacRae, 1996). For additional discussion of processes involved in stream channel enlargement due to urban runoff, see USEPA (1997) and CWP (2000).

Another contributing factor is the relatively low suspended sediment load in urban runoff. During centuries of intensive agriculture, sediment delivery rates were very high in the eastern U.S. This caused substantial aggradation as the sediments were deposited in floodplains. Following adoption of conservation practices and conversion of upland areas to fallow or urban land uses in the last half century, upland sediment losses have been substantially reduced (Ferguson, 1996; Ruhlman and Nutter, 1999). As a result, the excess kinetic energy formerly used to transport the sediment is presently available to entrain previously deposited bank and bed sediments. This phenomenon of "hungry" streams amplifies the process of stream incision and bank erosion (Heede, 1986). Gravel bars and riffle areas are buried under the sediments as banks erode (WDE, 1992). It is thought that one half of all suspended sediment in urban stream flow is thus generated by these processes of bank erosion (Yu and Wolman, 1986, as cited in MacRae, 1991). In the easily eroded soils of the San Diego Creek watershed, California, bank erosion has been identified as the source of nearly all the suspended sediments found in the streams (Geosyntec, 2002).

There are other adverse hydrological impacts of urbanization due to increased runoff. In Maryland watersheds, Klein (1979) noted that baseflow decreases as extent of urbanization increases. Ferguson and Suckling (1990) noted a similar relation in a Georgia Piedmont stream. On Long Island, Spinello and Simmons (1992, as cited in CWP, 1995a) noted substantial decreases in base flow in intensely urbanized watersheds. Other reviewers suggest that impervious surfaces may have less impact upon baseflow (WEF and ASCE, 1996). This may be due to the fact that although there may be less recharge, there is also is less evapotranspiration from urban sites, so the net effect is not as large as would be expected.

Klein (1979) also noted that summertime water temperatures increased in urban streams. He attributed this to the absence of shade since channels were wider and shallower, and to the reduction in relatively cool baseflow inputs. Urban runoff from paved surfaces and rooftops can be very warm, eliminating cold-water species that are intolerant of warm temperatures (Galli, 1990). Elevated temperatures during summer low flow conditions in urban streams can drive down the concentration of dissolved oxygen to very low levels, resulting in displacement of intolerant species by rough fish that can endure the conditions. In extreme cases during summertime droughts, dissolved oxygen levels can fall so low as to not even support aquatic life, resulting in fish kills. Warmer temperatures also accelerate the release of soluble fractions of zinc, copper, and PAHs into the water column from the sediment pool, further stressing benthic communities and fish.

2.5 IMPACTS OF URBAN RUNOFF HYDROLOGY ON BENTHIC COMMUNITIES

There is a considerable literature documenting the impacts of these changes due to urban land uses upon the resulting benthic macroinvertebrate communities and fisheries. Sedimentation impacts on fisheries can be severe, as the natural sequences of riffles and pools are lost, eliminating spawning areas (WDE, 1992), and reducing salmonid populations (Moscrip and Montgomery, 1999). Siltation in streams is now implicated as the leading cause of impairment to streams (USEPA, 1998).

In Washington, Booth and Jackson (1997) noted substantial declines in fish habitat as effective impervious area (EIA, the impervious areas piped directly to streams) exceeded 8% to 10%. Sediments have been shown to reduce the growth and fecundity of benthic macroinvertebrates, with complete mortality when deposits exceed 10 mm (Sweeney, 1993). Increasing the velocity of flow above 50 cm/sec also eliminates crayfish, resulting in the proliferation of mat-forming algae (Hart, personal comm.), greatly reducing the diatom supply for the remaining invertebrates. As a result, with the adapted food supply and spawning areas greatly reduced or eliminated, desirable native species die off or move away, to be replaced by undesirable opportunistic exotic species (USEPA, 1984; WDE, 1992: and sources cited in Schueler, 1987).

Klein (1979) noted that the species diversity index in streams declined in proportion to the extent of impervious area in the watershed. He attributed the decline to channel enlargement, lowered base flow between storms, and increased temperatures. He also noted the influence of migration barriers and the potential effects of toxic pollutants. He concluded that these effects may be avoided if TIA remains below 15%, or 10% for sensitive stream ecosystems that sustain trout. Numerous other studies have observed this threshold phenomenon of benthic impacts increasing in relation to the extent of watershed imperviousness (eg., Maxted and Shaver, 1997).

In Washington, channel incision and bedload resuspension due to urban runoff has been implicated as the major factor in the destruction of aquatic habitats (WDE, 1992). In the Piedmont area of Pennsylvania, sediments are the foremost pollution problem in aquatic systems (Sweeney, 1993). This can have far reaching consequences, leading to the impairment of economically important fisheries (CPB, 1992). Pederson and Perkins (1986) noted a decline in diversity due to urbanization as benthic taxa shifted from runoff intolerant shredders to runoff tolerant worms. They attributed this shift to the dominance of a silty erosional/depositional substrate in the urban streams, as well as to the absence of leaves, which were rapidly swept away by the increased flooding in the urban flow regime. Horner and others, (1996) noted a considerable reduction in benthic diversity as TIA approached 10%. In Mississippi, Shields and others (1994) noted a similar decline in habitat due to incision.

In Delaware, Maxted (1996) noted that the benthic community index declined substantially once 10% to 15% of a watershed was urbanized. The extent of this impairment was closely correlated with habitat index based upon observations of bank stability, width/depth ratios, point bars, and other evidence of erosion processes (Maxted and Shaver, 1997, 1999).

2.6 MITIGATION OF RUNOFF IMPACTS BY NONSTRUCTURAL BMPs

The preceding discussion emphasizes the urgent need to reduce the impacts of urban runoff. For too long, the toxic and hydrological consequences of urban runoff have been ignored, with unconscionable impacts upon the receiving waters. Even when water quality structural BMPs were finally mandated in 1991, they did not seem to mitigate the decline in benthic macroinvertebrate communities in the receiving waters (Maxted and Shaver, 1997, 1999). Few studies have investigated the impact of the BMPs on receiving waters, but it is thought that warm, algae laden effluent from wet ponds alters the composition of the benthic community in the receiving stream (Jones and others, 1997, and sources cited therein). Furthermore, thermal effects on streams receiving effluent from unshaded wet ponds can be substantial (Van Buren and others, 2000). A recent review by Horner and others (2000) also suggests that structural BMPs are not as effective as a continuous riparian buffer of native vegetation. This is supported by the findings of Zucker and White (1996), where instream biological metrics were correlated with extent of forested buffers.

However, structural pond BMPs do seem to provide habitat protection benefits, (Maxted and Shaver, 1999), so they are definitely an improvement over no BMP at all. In recognition of the need for further improvement, DNREC has formulated the Green Technology methodology to provide a comprehensive approach to the problem.

By using nonstructural BMPs that filter and settle out pollutants in linear landscaped features that provide for tree cover, algal and thermal impacts can be minimized in comparison to large ponds. Furthermore, by integrating these nonstructural BMPs into the landscape, it is possible to provide for more infiltration than is possible for a pond placed at the lowest point in a site. This can have substantial benefits in terms of reducing base flow temperatures, as discussed above. A final benefit is the potential for a nonstructural BMP to be a landscaped amenity, instead of a large isolated structure requiring substantial area for ancillary access, buffering, screening and maintenance facilities.

By defining the problem of urban runoff impacts in such detail, the way to mitigate these impacts becomes more focused. However, to properly address pollutant load impacts, it is necessary to have a realistic idea of the extent of pollutant loads that can be anticipated from the various urban land cover categories. Chapter 3 examines the literature on pollutant loads from such land covers so as to project the most likely loads of pollutants in urban runoff.

CHAPTER 3 URBAN RUNOFF LOADING

3.1 RUNOFF POLLUTANT LOADING OVERVIEW AND MODEL APPROACH

Urban runoff pollutants comprise many different types of chemical compounds, as discussed in Chapter 2. Runoff from parking lots and streets has been shown to have a greater correlation with impacts on urban streams than that from roofs (Coughlin and Hammer, 1973). Toxic metals such as lead, copper, and zinc associated with vehicular uses have been found in 96% of the samples in the metropolitan Washington area (MDE, 1986). Although newer urban areas are typically minor sediment generators, decaying pavement contaminated with particles from tire wear can be the dominant component in runoff in older urban sites (Myers and others, 1985; Harper, 2002).

The pollutants in urban runoff of most concern are total suspended sediments (TSS), total phosphorus (TP), and total nitrogen (TN). Copper and zinc seem to be the most prevalent metals found at toxic levels (Cooke and others, 1994). Most metals and the particulate fractions of nutrients have a high affinity with suspended sediment, so methods to reduce TSS loadings will also tend to reduce loadings of these compounds. For this reason, nitrogen and phosphorus have been segregated into soluble and particulate fractions.

Settling in bioretention facilities, and filtration in filter strips and biofiltration swales are effective methods for treating particulate pollutants. However, for soluble toxic metals such as soluble fractions of zinc and copper, settling is not effective. Zinc can be quite soluble at the pH of urban runoff, as are dissolved copper, volatile organic compounds (VOCs), and many PAHs. These pollutants from “hot spots” such as automotive service centers, certain industrial sites, and high intensity commercial sites such as convenience stores are not removed by settling processes. For these soluble toxic pollutants, bioretention facilities, and/or a treatment train approach is necessary.

Depending upon the land cover in the source area, different amounts of various pollutants will be washed off during runoff events. Using values for average annual mass loads by land cover category alone can introduce substantial error for pervious surfaces, since the volume of runoff varies greatly in response to soil type. This variation is quite substantial at the low rainfall volumes generated by the quality storm discussed in Chapter 4.

To better estimate pollutant loading by different land covers according to their respective areas, it is thus more accurate to delineate expected pollutant loading from each category of urban land cover in terms of its area weighted event mean concentrations (EMCs). The weighted EMC values can then be multiplied by the runoff volume from each category of urban land cover polygons. This product represents the mass load for each individual combination of land cover/soil type during a runoff event. This method accounts for the variation in runoff volumes generated from the extensive combinations of land covers and soil types discussed in Chapter 5.

However, EMCs from differing land cover categories vary widely from event to event, from region to region, and from study to study. When pollutants have accumulated after periods of dry weather, the earliest runoff often has elevated pollutant levels, while EMCs from runoff later in the event or from subsequent events are much lower (Novotny and others, 1994; Soeur and others, 1994). Although less prevalent in events from large basins, (Characklis and Weisner, 1997), this “first flush” phenomenon is often found at the site scale addressed by this Manual. Examination of the literature also indicates that concentrations are generally higher in the smaller events, or events that occur during drier years. EMC values thus can vary over several orders of magnitude between the highest and lowest values observed (see data presented Waschbush and others, 2000). Therefore, continuous simulation models such as SLAMM and PCSWMM use Monte Carlo methods based upon a random number generator to generate a range of loading values that changes from event to event, and take into account pollutant accumulation and washoff functions based upon interevent intervals.

Since DURMM is based upon a single event approach, the option to use a range of values is not available. If the intent is to replicate a “typical” event, it is essential to derive the proper value for the EMC. The arithmetic mean of observations is often reported in many studies. More recent studies report the geometric mean as well, since it weights extreme values less, and thus reveals the central tendency better than the arithmetic mean. However, for the purposes of a single event model, the best approach is to add up the total loads generated over a year, and divided by the annual runoff volume. This “flow-weighted” mean thus reflects the “average” event. By using an annual flow-weighted mean, the inherent variability between events is balanced out over the course of a year.

The runoff volume approach discussed in Chapter 5 can be used to determine runoff volumes of individual land uses to obtain the flow-weighted mean. Where annual loads and runoff volumes are supplied in several studies, flow-weighted means were able to be determined, as indicated in Tables 3-1 through 3-4. Tables 3-1 through 3-4 include these flow-weighted means along with the arithmetic and geometric means reported in the literature. This approach not only provides another reasonable estimate for the typical EMC, it further reinforces the central tendencies. The arithmetic, geometric and flow-weighted means reported in the studies were then used as the basis for establishing the values used in DURMM. Values were adjusted in relation to the geometric mean as discussed below if warranted by further examination.

Total impervious loads are determined by summing up the product of event runoff, times the EMC from each category of impervious area, times area of each category. A similar approach is taken for pervious loads. Total pollutant load is then the sum of impervious and pervious loads. Mass loading thus estimated from the pervious and impervious surfaces is added together to determine total mass loading, which is then divided by total runoff volume to provide the EMC from the site. To the extent that impervious disconnection (discussed in Chapter 5) reduces total runoff, the total loads are then reduced proportionately. EMCs into the BMP are then determined by total load divided by total runoff.

3.2 RUNOFF POLLUTANT TRANSPORT PROCESSES

Note that the EMC values addressed in this Manual are exclusively allocated for surface runoff concentrations. In impervious areas, overland flow dominates the runoff response. However, in largely pervious areas, up to 80% of all annual runoff can occur through subsurface flow pathways, ending up as recharge to base flow (Correll and others, 1997). This situation is typical of watersheds throughout the East, where most streamflow comprises groundwater that has infiltrated previously. Therefore, subsurface pollutant loads can be an important contribution to total annual loads, particularly in agricultural settings.

One of the most effective BMPs is impervious area disconnection, where flow from impervious areas is directed over lawns and other pervious surfaces. For most pollutants, EMCs from impervious surfaces will be substantially reduced where infiltrated through a vegetated root zone overlying an intact soil profile. Vegetative uptake and microbial immobilization transform and sequester nutrients, toxic metals and PAHs. However, once past the root zone, highly soluble pollutants that are not adsorbed by soils can pass into groundwater relatively unaltered.

Parmer and others (1995) reviewed the literature in terms of the potential impacts of urban runoff constituents to groundwater. Nitrate is the most soluble nutrient, while dissolved zinc is the most soluble metal. Depending upon half-life and adsorption coefficients, many pesticides can also infiltrate into groundwater. Organophosphate pesticides are less persistent, but less likely to be adsorbed than organochlorine pesticides. Road salts are very soluble, and minimal reductions in runoff EMCs are observed in BMPs.

Therefore, if heavily polluted runoff is treated by underground infiltration facilities without extensive pretreatment, there is a real concern for groundwater pollution. This is an important consideration for soluble metals and PAHs (Pitt and others, (1996). Nitrate is also very soluble and has minimal uptake by mineral soils. Phosphorus is generally readily bound within the soil profile, even under the very high loads of septic systems (see sources cited in Gold and Sims, 2000). The extent of phosphorus and metals adsorption is quite dependent upon soil properties, as discussed in more detail for bioretention facilities in Chapter 11.

Where infiltration dominates the runoff response, nitrate leaching losses can comprise the majority of total nutrient loading from urban runoff. At present, there are relatively few studies of nitrate leaching losses in urban settings. Most of the literature on urban runoff EMCs focuses on overland flow collected by weirs and other types of collectors. However, there are several that examine leaching losses of nutrients from turf.

Unlike agricultural crops, which are plowed up annually, the well-established root systems of lawns are effectively permanent, so nutrients remain bound up in the shallow soil profile. As a result of nitrate being taken up and/or immobilized in the root zone of the grasses, these studies report minimal leaching losses under normal fertilization and irrigation practices. Average annual flow-weighted loss for typical turf has been reported as low as 0.21 mg/l (Gold and others, 1990) to 1.06 mg/l (Geron and others, 1993). An EPIC simulation of soils thoroughly irrigated with well water at 6.3 mg/l projected a flow-weighted mean of 0.48 mg/l for heavily fertilized fairways (King and Balogh, 1999). Concentrations from over-fertilized sandy soils under excessive irrigation can be as high as 45 mg/l (Watts and others, 1993), but this is not representative of the typical situation.

It is important to note that the volume of rainfall infiltrated into urban lawns typically exceeds the volume that runs off by a factor of at least two (King and Balogh, 1999) or over ten (Kussow, 2002). Several studies show very little runoff from turf, unless frozen (Kussow, 2002) or previously saturated, under high rainfall depths or intensities (Gross and others, 1990; Cole and others, 1997). Depending upon the soil type and climate, leaching loads vary from one-third the runoff losses (King and Balogh, 1999) to over four times the runoff losses (Linde and Watschke, 1997).

Given this variability, and the difficulty in establishing an approach to replicate such interactions, consideration of subsurface losses is beyond the scope of the surface runoff processes addressed in this Manual. DURMM takes the oversimplified approach of omitting groundwater loading in its entirety. However, since such subsurface losses can be substantial, they should be recognized in selecting the appropriate BMP.

For these reasons, the approach in this Manual is to provide a quantitative estimate of total pollutant loads in surface runoff according to their EMCs, so as to identify the location and degree of potential impacts, and provide direction as to the most appropriate approach to mitigate these impacts. By comparing loading EMCs, receiving water criteria, and site constraints, the proper choice of BMPs becomes evident. For instance, infiltration trenches would be discouraged where EMCs of nitrate or soluble metals are high, while bioretention facilities would be recommended. DURMM thus provides for designers to focus on the optimal BMPs by taking into account implications of individual land cover categories and their runoff response. This approach advances BMP design beyond that offered by a generalized land use approach and simplified runoff volume estimates.

3.3 URBAN RUNOFF POLLUTANT LOADING ESTIMATES

Given that this approach is to segregate and estimate runoff EMCs by category of pervious and impervious urban land covers, the literature has been reviewed to determine the most appropriate value to allocate for each type of land cover. However, the literature on EMCs by land cover type is quite thin. There have been numerous studies of different land covers aggregated into certain land uses, but most do not provide information on which land cover (roofs, parking, loading, streets, sidewalks, lawns, landscaping, etc.) is responsible for which proportion of the total load.

Tables 3-1 through 3-5 summarize much of the literature published on urban runoff pollutant loading by land cover. Tables 3-1 and 3-2 summarize TSS loading and phosphorus species loading, and Tables 3-3 and 3-4 summarize nitrogen species loading. Table 3-5 summarizes Copper and Zinc loads, along with information on the location and method of the sources, and where flow weighted data has been included. Agricultural land cover categories are included in these tables since they represent pre-development conditions.

Note that the TSS value allocated for lawns is a relatively high value of 125 mg/l. While TSS EMCs reported from vegetative filters and biofiltration swales can be as low as 4 mg/l, the higher value represents a “typical” urban lawn that may have some bare patches. A lower value could be justified for fairways or well maintained lawns. Likewise, there is a considerable variation in TSS values from streets. The value in Table 3-1 is close to the observed geometric mean, but much lower than the 340 mg/l needed to calibrate the SLAMM model (Waschbusch and others, 2000). Surprisingly, low volume streets seem to have the higher TSS EMCs, and driveways have the highest EMCs of all impervious surfaces. Since the flow weighted TSS varied from 54 mg/l for low tree canopy streets to 211 mg/l for high tree canopy streets, this trend may reflect TSS contributions by adjacent street trees.

Phosphorus loadings in Tables 3-1 and 3-2 are segregated into soluble and insoluble forms so as to segregate the fractions of total phosphorus loads that settle out from that which remains dissolved. Note that most observations provided only total and dissolved values, so EMCs of the particulate fraction has been calculated by subtraction.

Surprisingly high values are reported for phosphorus losses from woods, even though the undisturbed environment of a well-established litter layer would suggest that little phosphorus would be lost in runoff. (Note that the volume of annual runoff from woods is usually so low that annual loads remain very low.) However, Garn (2002) noted that high levels were found in runoff from wooded sites next to lawns. Waschbusch and others (2000) also note a strong correlation between phosphorus EMCs on streets and the extent of tree cover along streets. In urban streets, decomposition pathways that normally recycle nutrients do not occur. Instead, after leaves fall on impervious surfaces, they are broken down by largely mechanical processes and pass into urban runoff relatively unaltered. Table 3-2 also shows that EMCs of dissolved phosphorus from lawns in a basin with extensive tree cover was almost twice that of the lower canopy basin.

For this reason, soluble phosphorus loading from landscaping is allocated value of 1.10 mg/l, twice the 0.55 mg/l assigned to lawns. Likewise, particulate phosphorus from landscaping is allocated at 2.50 mg/l, over three times that from lawns. Using such a high value provides a mechanism, although over-simplified, to account for the effect of phosphorus loads from landscaping upon adjacent categories without having to adjust their EMCs directly. Instead, the designer can allocate the area of landscaping as a separate category from the underlying lawns.

Total nitrogen loads are partitioned into nitrate, ammonium and organic nitrogen loads. TKN is not addressed, as it is the sum of ammonium and organic loads. TN and TKN loads are not addressed individually. Note that many observations only provide partial measurements, so EMCs of the remaining species have been calculated by subtraction.

Since vegetation is absent in impervious surfaces, most of the nitrogen species in runoff from impervious surfaces would seem to be generated from atmospheric deposition. Averaged over a 17 year period at Wye, MD, Correll and others (1994) noted atmospheric inputs of 5.56 kg/ha/yr of nitrate-N, 3.18 kg/ha/yr of ammonium-N and 3.62 kg/ha/yr of organic nitrogen-N. Note that organic N load was 41% of the nitrate and ammonium loads. Given average annual rainfall of 1.08 meters, these values represent 0.51, 0.29 and 0.34 mg/l, respectively.

Table 3-1: Mean TSS and Total Phosphorus EMCs in Urban and Agricultural Runoff (mg/l)

	TOTAL SUSPENDED SOLIDS												TOTAL PHOSPHORUS											
SOURCE	FLAT ROOFS	PITCHED ROOFS	PARKING LOTS	MEDIUM STREETS	DRIVE-WAYS	LAWNS	LAND-SCAPE	WOODS	CONV. TILL	CHISEL PLOW	NO- TILL	PAS-TURE	FLAT ROOFS	PITCHED ROOFS	PARKING LOTS	MEDIUM STREETS	DRIVE-WAYS	LAWNS	LAND-SCAPE	WOODS	CONV. TILL	CHISEL PLOW	NO- TILL	PAS-TURE
Owens et al, 1983												160												
Polls and Lanyon, 1976				266				34																
Peterjohn and Correll, 1984									6480												5.03			
Laflen and Tabatabai, 1984									18940	9710	4940													
Langdale et al, 1985									2310	970											0.08	0.17		
Correl et al, 1984																				0.81	0.16			0.56
Gilliam et al, 1993a									4111												1.70			
Gilliam et al, 1993b									4103												2.14			
Correl et al 1994																				0.35		2.32		0.81
Mendez et al, 1999									7890															
Linde and Watschke, 1997						15																		
Gross et al, 1991						231																		
Gross et al, 1990						25																		
Gross et al, 1990						8																		
Horner et al, 1994			45					1							0.08					0.10				
Garn, 2002																		2.06	3.52					
Schueler & Shepp 1993			11	3											0.50	0.06								
Pitt et al 1996a				450	310	118										0.30	0.63	0.29						
Pitt et al 1996b		0		136	687	807								0.04		0.49	0.62	0.20						
Pitt, et al, 1996	3	27	16	15			38																	
DSWF, 1996	9	19	27	172	173	602	37						0.20	0.09	0.45	0.63	1.16	1.67						
Wisconsin, 1992	19	36	474	241	193	457							0.24	0.19	0.48	0.53	1.50	3.47						
Bannerman et al, 1993	15	27	58	326	173	397							0.20	0.15	0.19	1.07	1.16	2.67						
Steuer et al, 1997	24	36	138	305	157	262							0.09	0.06	0.20	0.23	0.35	2.33						
Waschbusch et al, 1999a	18	16	51	69	34	91							0.07	0.11	0.05	0.32	0.18	1.20						
Waschbusch et al, 1999b		20		211	68	128								0.16		0.76	0.24	1.54						
Waschbusch et al, 1999c	21	18	75	94	266	77							0.13	0.07	0.11	0.38	0.50	1.05						
Waschbusch et al, 1999d	21	20	75	94	255	88							0.13	0.12	0.11	0.38	0.47	1.13						
Arithmetic Mean	16	22	97	183	232	236	37	18	7306	5340	4940	160	0.15	0.11	0.24	0.47	0.68	1.60	3.52	0.42	1.82	1.24		0.68
Geometric Mean	14	14	54	111	177	121	37	6	5791	3069	4940	160	0.14	0.10	0.18	0.38	0.55	1.23	3.52	0.30	0.74	0.63		0.67
MODEL VALUE	15	20	60	110	180	125	50	30	6000	3000	1000	160	0.15	0.11	0.25	0.38	0.49	1.30	3.60	0.30	2.30	1.70	1.10	1.30

Table 3-2: Mean Particulate and Soluble Phosphorus EMCs in Urban and Agricultural Runoff (mg/l)

	PARTICULATE PHOSPHORUS												SOLUBLE PHOSPHORUS											
SOURCE	FLAT ROOFS	PITCHED ROOFS	PARKING LOTS	MEDIUM STREETS	DRIVE-WAYS	LAWNS	LAND-SCAPE	WOODS	CONV. TILL	CHISEL PLOW	NO- TILL	PAS-TURE	FLAT ROOFS	PITCHED ROOFS	PARKING LOTS	MEDIUM STREETS	DRIVE-WAYS	LAWNS	LAND-SCAPE	WOODS	CONV. TILL	CHISEL PLOW	NO- TILL	PAS-TURE
Peterjohn and Correll, 1984									4.82												0.21			
Polls and Lanyon, 1976																0.07				0.04				
Laflen and Tabatabai, 1984																					0.08	0.17	0.60	
Langdale et al, 1985									1.82	0.85											0.13	0.23		
Gilliam et al, 1993a									1.16												0.29			
Gilliam et al, 1993b									1.70												0.44			
Correl et al 1994								0.29		2.27		0.76								0.06		0.05		0.05
Cole et al, 1997																		5.21						
Linde and Watschke, 1997																		3.19						
Gross et al, 1991																		0.06						
Gross et al, 1990																		0.03						
Kussow, 2002																		0.12						
Garn, 2002						1.58	2.48											0.48	1.04					
Schueler & Shepp 1993			0.34	0.05											0.16	0.01								
Pitt et al 1996a				0.24	0.25	0.09										0.06	0.38	0.20						
Pitt et al 1996b		0.02		0.46	0.60									0.02		0.03	0.02	0.66						
Wisconsin, 1992	0.13	0.11	0.41	0.39	0.63	1.07							0.11	0.08	0.07	0.14	0.87	2.40						
Bannerman et al, 1993	0.12	0.09	0.14	0.76	0.67	1.22							0.08	0.06	0.05	0.31	0.49	1.45						
Steuer et al, 1997	0.06	0.04	0.18	0.22	0.31	2.24							0.03	0.02	0.02	0.01	0.04	0.09						
Waschbusch et al, 1999a	0.11	0.06	0.08	0.20	0.11	0.63							0.02	0.05	0.02	0.12	0.07	0.57						
Waschbusch et al, 1999b		0.09		0.42	0.12	0.71								0.09		0.34	0.12	0.85						
Waschbusch et al, 1999c	0.09	0.04	0.09	0.20	0.36	0.56							0.02	0.03	0.03	0.19	0.14	0.49						
Waschbusch et al, 1999d	0.09	0.06	0.09	0.20	0.34	0.58							0.02	0.07	0.03	0.18	0.14	0.55						
Arithmetic Mean	0.10	0.06	0.19	0.31	0.38	0.96	2.48	0.29	2.37	1.56		0.76	0.05	0.05	0.06	0.13	0.25	1.09	1.04	0.05	0.23	0.15		
Geometric Mean	0.10	0.06	0.16	0.25	0.31	0.73	2.48	0.29	2.04	1.39		0.76	0.04	0.04	0.04	0.08	0.14	0.47	1.04	0.05	0.19	0.12		
MODEL VALUE	0.10	0.06	0.20	0.30	0.35	0.75	2.50	0.25	2.10	1.40	0.70	0.75	0.05	0.05	0.05	0.08	0.14	0.55	1.10	0.05	0.20	0.30	0.40	0.55

Table 3-3: Mean Total and Nitrate Nitrogen EMCs in Urban and Agricultural Runoff (mg/l)

	TOTAL NITROGEN												NITRATE NITROGEN											
SOURCE	FLAT ROOFS	PITCHED ROOFS	PARKING LOTS	MEDIUM STREETS	DRIVE-WAYS	LAWNS	LAND-SCAPE	WOODS	CONV. TILL	CHISEL PLOW	NO- TILL	PAS-TURE	FLAT ROOFS	PITCHED ROOFS	PARKING LOTS	MEDIUM STREETS	DRIVE-WAYS	LAWNS	LAND-SCAPE	WOODS	CONV. TILL	CHISEL PLOW	NO- TILL	PAS-TURE
Peterjohn and Correll, 1984									4.45												4.45			
Laflen and Tabatabai, 1984																					0.18	0.21	1.59	
Correl et al, 1984								0.36	5.26			0.27								0.36	5.26			0.27
Langdale et al, 1985									5.04												5.04			
Gilliam et al, 1993a									0.90												0.90			
Gilliam et al, 1993b									0.83												0.83			
Owens et al, 1983																								1.95
Owens et al, 1989																								0.80
Correl et al 1994								0.14				0.40								0.14		1.61		0.40
Mendez et al, 1999									27.89															
Cole et al, 1997																		2.09						
Linde and Watschke, 1997																		0.42						
King and Balogh, 1997																		2.71						
Gross et al, 1990																		0.27						
Gross et al, 1990																		0.04						
Horner et al, 1994			0.32				0.03								0.32				0.03					
Kussow, 2002																		0.12						
Garn, 2002																		0.12						
Schueler & Shepp 1993			0.01	0.92											0.01	0.92								
Pitt et al 1996a																								
Pitt et al 1996b																								
Steuer et al, 1997	0.49	0.46	0.34	0.32	0.30	0.40							0.49	0.46	0.34	0.32	0.30	0.40						
Arithmetic Mean	0.49	0.46	0.22	0.62	0.30	0.40	0.03	0.25	7.40			0.34	0.49	0.46	0.22	0.62	0.30	0.77	0.03	0.25	2.78	0.91	1.59	0.86
Geometric Mean	0.49	0.46	0.10	0.54	0.30	0.40	0.03	0.23	3.68			0.33	0.49	0.46	0.10	0.54	0.30	0.33	0.03	0.23	1.59	0.58	1.59	0.64
MODEL VALUE	0.50	0.45	0.30	0.55	0.30	0.35	0.25	0.25	1.60	1.80	2.00	0.60	0.50	0.45	0.30	0.55	0.30	0.35	0.25	0.25	1.60	1.80	2.00	0.60

Table 3-4: Mean Ammonia and Organic Nitrogen EMCs in Urban and Agricultural Runoff (mg/l)

	AMMONIA NITROGEN												ORGANIC NITROGEN											
SOURCE	FLAT ROOFS	PITCHED ROOFS	PARKING LOTS	MEDIUM STREETS	DRIVE-WAYS	LAWNS	LAND-SCAPE	WOODS	CONV. TILL	CHISEL PLOW	NO- TILL	PAS-TURE	FLAT ROOFS	PITCHED ROOFS	PARKING LOTS	MEDIUM STREETS	DRIVE-WAYS	LAWNS	LAND-SCAPE	WOODS	CONV. TILL	CHISEL PLOW	NO- TILL	PAS-TURE
Peterjohn and Correll, 1984									1.89															
Laflen and Tabatabai, 1984									0.19	0.58	1.23													
Correl et al, 1984								0.98	1.88			0.32								2.17	1.65			1.82
Langdale et al, 1985																								
Gilliam et al, 1993a									0.02												3.43			
Gilliam et al, 1993b									0.34												4.27			
Owens et al, 1983												6.00												3.55
Owens et al, 1989												1.60												2.70
Correl et al 1994								0.12		0.18		0.15								1.39		2.97		3.16
Mendez et al, 1999									4.30												23.59			
Cole et al, 1997						3.65																		
Linde and Watschke, 1997						*0.32*												*1.13*						
Gross et al, 1990						0.21																		
Gross et al, 1990						0.07																		
Horner et al, 1994			0.22												0.35				0.16					
Garn, 2002						0.96																		
Schueler & Shepp 1993			1.58	0.19											3.36	0.65								
Pitt et al 1996a				0.05	0.10	0.40										1.75	2.50	0.80						
Pitt et al 1996b		0.10		0.05	0.30	0.50								0.70		1.55	0.80	0.80						
Steuer et al, 1997	0.67	0.44	0.22	0.35	0.12	0.26							0.93	0.56	1.38	0.95	1.68	9.04						
Arithmetic Mean	0.67	0.27	0.67	0.16	0.17	0.80		0.55	1.44	0.38	1.23	2.02	0.93	0.63	1.70	1.23	1.66	2.94	0.16	1.78	8.23			2.81
Geometric Mean	0.67	0.21	0.42	0.11	0.15	0.41		0.35	0.52	0.32	1.23	0.82	0.93	0.63	1.18	1.14	1.50	1.60	0.16	1.73	4.89			2.72
MODEL VALUE	0.65	0.25	0.45	0.15	0.15	0.50	0.45	0.40	0.60	1.00	1.70	0.70	0.95	0.65	1.20	1.15	1.50	1.80	1.75	1.75	4.90	5.40	6.00	2.75

Table 3-5: Mean Copper and Zinc EMCs in Urban Runoff (mg/l), Sources

TOTAL COPPER (ug/l)												
SOURCE	FLAT ROOFS	PITCHED ROOFS	PARKING LOTS	STREETS	DRIVE-WAYS	LAWNS	LAND-SCAPE	WOODS	CONV. TILL	CHISEL PLOW	NO- TILL	PAS-TURE
CH2MHill, 2000				22.1		5.7		5.3	5.4			5.4
Omni source						9.8			11.3			
Horner et al, 1994			4.5									
Tiefefthaler et al, 2001			28.0									
Cook et al, 1996						10.0						
Pitt, et al, 1996	5.0	46.0	285.0	10.0								
DSWF, 1996	7.0	20.0	51.0	24.0	17.0	17.0						
Wisconsin, 1992	10.0	5.0	21.0	25.0	20.0	13.0						
Bannerman et al, 1993	9.0	15.0	15.0	56.0	17.0							
Steuer et al, 1997	20.0	7.0	22.0	30.0	34.0							
Arithmetic Mean	10.2	18.6	60.9	27.9	22.0	11.1		5.3	8.4			5.4
Geometric Mean	9.1	13.7	27.8	24.6	21.1	10.4		5.3	7.8			5.4
MODEL VALUE	10	15	30	25	25	15	5	5	10	10	10	5

TOTAL ZINC (ug/l)												
SOURCE	FLAT ROOFS	PITCHED ROOFS	PARKING LOTS	STREETS	DRIVE-WAYS	LAWNS	LAND-SCAPE	WOODS	CONV. TILL	CHISEL PLOW	NO- TILL	PAS-TURE
CH2MHill, 2000				214.6		25.4		24.8	23.5			23.5
Omni source						75.8			64.0			
Horner et al, 1994			90.0									
Tiefefthaler et al, 2001			293.0									
Cook et al, 1996						200.0						
Pitt, et al, 1996	181.0	476.0	64.0	38.0								
DSWF, 1996	256.0	312.0	139.0	173.0	107.0	50.0						
Wisconsin, 1992	363.0	153.0	249.0	245.0	113.0	60.0						
Bannerman et al, 1993	330.0	149.0	178.0	339.0	107.0							
Steuer et al, 1997	348.0	201.0	178.0	166.0	148.0							
Arithmetic Mean	295.6	258.2	170.1	195.9	118.8	82.2		24.8	43.8			23.5
Geometric Mean	286.5	232.6	151.7	164.0	117.6	64.9		24.8	38.8			23.5
MODEL VALUE	290	240	170	160	120	90	25	25	40	50	60	25

SOURCE	LOCATION, METHOD, COMMENTS
Peterjohn and Correll, 1984	MD- arithmetic means, natural events
Laflen and Tabatabai, 1984	IA- arithmetic mean, simulated events
Correl et al, 1984	MD- arithmetic mean, natural events
Langdale et al, 1985	GA- arithmetic means, natural events
Gilliam et al, 1993a	NC- geometric means, natural events
Gilliam et al, 1993b	NC- flowweighted means, natural events
Correl et al 1994	MD- arithmetic means, buffered stormflow
Mendez et al, 1999	VA- arithmetic means, simulated events
Cook et al, 1996	CA- arithmetic means,, streamflow measurments
Cole et al, 1997	Oklahoma, flow weighted mean, intense simulated
Linde and Watschke, 1997	Oklahoma-flow weighted mean, natural and simulated
Gross et al, 1991	Maryland- flow weighted mean, simulated rainfall
Gross et al, 1990	Maryland- flow weighted mean, dry year events
Gross et al, 1990	Maryland- flow weighted mean, average year events
Horner et al, 1994	Nationwide summary
Kussow, 2002	Wisconsin- flow weighted mean, natural events
Garn, 2002	Wisconsin- average of geometric mean, several events
Schueler & Shepp 1993	Maryland -arithmetic mean, natural events
Pitt et al 1996a	Toronto-arithmetic mean, winter events
Pitt et al 1996b	Toronto -arithmetic mean, warm season events
Pitt, et al, 1996	Alabama- arithmetic mean, natural events
DSWF, 1996	Wisc. & Ala.- arithmetic mean, natural events
Wisconsin, 1992	Wisconsin- geometric mean, several events
Bannerman et al, 1993	Wisconsin- geometric mean, several events
Steuer et al, 1997	Michigan- geometric mean, several events
Waschbusch et al, 1999a	Wisconsin-average geometric mean, many events in two basins
Waschbusch et al, 1999b	Wisconsin- flow weighted mean, many events, wooded basin
Waschbusch et al, 1999c	Wisconsin- flow weighted mean, many events, open basin
Waschbusch et al, 1999d	Wisconsin- flow weighted mean, many events, both basins
CH2MHill, 2000	Recommended EMCs for NC based upon measurments
Tiefefthaler et al, 2001	California, mean of many rainfall simulator events

The Model of the Chesapeake Airshed calls for atmospheric deposition rates of 3.22 and 1.94 kg/ha/yr of nitrate-N and ammonia-N, respectively in Wye, MD (Linker and others, 2000). Based upon the results of Scudlark and Church (1993, as cited in Horsley and Witten 1998) for Lewes, DE, average EMCs for nitrate are 0.28 mg/l and 0.17 mg/l for ammonium. Using a 41% value for organic N, this suggests a value of 0.20 mg/l for the particulate organic component. Compared to the EMCs reported in Steuer and others (1997) and Pitt and others (1996), the inorganic forms of nitrogen predicted from deposition are slightly less than observed EMCs. However, levels of organic N in urban runoff from impervious surfaces seem to exceed atmospheric deposition by a factor of at least three. This suggests that there must be additional sources local to the urban environment. Given a TKN level as high as 9.30 mg/l from lawns (Steuer and others, 1997), lawns (and/or trees) would seem to be the primary source of organic N deposition on adjacent impervious surfaces.

For organic N, the geometric mean of the study residual values is applied to impervious surfaces. For lawns, the residual value of 9.04 from Steuer and others (1997) may represent an extreme when compared to the 0.80 reported by Pitt and others (1996), so a design value of 3.45 is used, based on the geometric mean of the studies. This value is similar to the 3.16 mg/l reported for pastures by Correll and others (1994). Given their turf cover and nutrient inputs, pastures could be considered similar to lawns in this analysis.

In the Rhode River watershed near Wye, MD, organic N from crops was 77% of total N in surface runoff, with 92% in the particulate form. In contrast, groundwater levels of organic-N were only 2.4% of total subsurface N loads (Peterjohn and Correll, 1984). Dillaha and others (1989) noted that 95% to 97% of organic N in runoff from plowed fields was in the particulate form. Organic N comprises 80% of the total nitrogen in stormflow from pastures, and over 50% of the load from crops and woods, while base flow ratios varied from 48% to 26% (Correll and others, 1995). These studies suggest that surface runoff pathways dominate stormflow losses of organic N. This was confirmed in subsequent study in the Maryland Piedmont by the same authors, although they reported that 70-80% of organic N was in a dissolved form (Jordan and others, 1997).

If organic N were particulate, most of the organic N in urban runoff would be in a form amenable to settling and filtration. However, organic N EMC reductions by vegetative filtration BMPs are variable, ranging from 30% to 80% (See Chapter 9). This implies that some of the organic N loads must be either soluble, or adsorbed into fractions too fine to settle with these BMPs. Parsons and others (1993) reported average organic N EMC reductions from 31% to 52% from filter strips below cropped areas. Since this study had events similar to conditions encountered in urban BMPs, the lower range seems more appropriate. It seems that organic nitrogen is present in a fraction too fine to filter thoroughly, but with a low ability to percolate into groundwater.

While the main thrust of DURMM is to address sediment and nutrient loads, there is a potential for toxicity from elevated levels of metals, PAHs, organic compounds, and petroleum hydrocarbons in runoff originating from many development sites. Petroleum hydrocarbons from "hot spots" such as intense industrial and commercial land uses can be quite elevated in comparison to residential streets (Shepp, 1996). A similar relationship was noted for copper and zinc (Schueler and Shepp, 1995). While there is little data on PAH loading by land cover, Steuer and others (1997) noted that total PAHs from parking lots was 90 µg/l, while other impervious surfaces were in the range of 1 to 5 µg/l. No PAHs were noted in runoff from lawns.

Since the literature is just now emerging, the approach taken in DURMM is to wait until better data is available for estimating PAH and organic compound loading from hot spots. Note that the soluble forms of metals are more toxic, but there are no nationwide standards for these species. Table 3-5 summarizes metals loading by land cover.

By defining the extent of urban runoff loads in this method, the requirements needed to mitigate these loads become more focused. However, to more fully address urban runoff impacts, it is also necessary to have a realistic idea of the hydrological impacts of urban runoff. Chapter 4 examines the criteria needed for pollutant removal, and relates these criteria to the Delaware's climate characteristics.

CHAPTER 4 URBAN RUNOFF CRITERIA

4.1 CRITERIA FOR URBAN RUNOFF POLLUTANT TREATMENT

The primary factor in developing treatment criteria for the control of pollutants in urban runoff is to define the runoff volume captured by a water quality BMP. For a treatment process to reduce pollutants to acceptable levels, a BMP must intercept enough of the annual loading of runoff. If the treatment volume is inadequate, it will not provide the necessary benefits that may be otherwise attainable.

The State of Maryland has established goals to reduce TSS loading from urban runoff by 80%, and reduce total phosphorus (TP) loading by 40% (MDE, 1999). It is thought that TSS removal will address most toxic pollutants such as metals, since they are generally associated with sediments. The 80% removal value for TSS implies that the great majority of the annual runoff volume must be captured by a BMP to attain the target level of treatment. In Maryland, roughly 70% of total rainfall occurs in events up to an inch, but an inch of treatment for the larger storms boosts the effective capture volume into the range of 80 to 85% (Prince Georges Co., 2000). Many authorities thus establish a treatment volume at one inch of rainfall as being adequate to attain the desired goals, while still being cost effective (Claytor and Schueler 1996; Prince Georges Co., 2000; MDE, 1999).

The Coastal Zone Management Program recommends treatment of the 2 year, 24 hour storm of over 3 inches (USEPA, 1991). Continuous modeling indicates that nearly as effective treatment could be provided more cost effectively by using a smaller design event, so Wisconsin recommends a design event in the range of 1.25 to 1.5 inches (WDNR, 1995). However, rules proposed to be adopted by the USEPA for marinas and agricultural areas require treatment of 80% of the annual runoff, not annual rainfall. Since pervious areas generate much less runoff than impervious areas at low rainfall amounts, the required rainfall volume to capture 80% of runoff will be greater in sites with more pervious area.

Table 4-1 displays the percentage of annual rainfall volume during events of the indicated size increment at Porter Reservoir in New Castle Co., Dover in Kent Co., and Georgetown in Sussex Co. (Leathers, 2000). Precipitation increment is the product of the average rainfall in each increment and its percent of total rainfall. Note that the final rainfall increment for extreme events is larger, since these events provide 0.3% to 0.6% of total annual rainfall. At rainfall increments greater than the recommended capture depth of 2.0 inches, captured precipitation is the product of capture depth times the incremental percentage of annual rainfall. The amount of annual rainfall captured by a specific volume is then determined by summing across the rows. Depending upon location, a treatment volume of 2.0 inches intercepts 95% to 97% of annual rainfall.

Table 4-1: Annual Precipitation Distribution and Runoff Capture Volumes of the 2.0 Inch Event, New Castle, Kent And Sussex Counties, Delaware

LOWER INCREMENT	0.00	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	TOTAL
UPPER INCREMENT	0.24	0.49	0.74	0.99	1.24	1.49	1.74	1.99	2.24	2.49	2.74	2.99	6.00	
PORTER RESERVOIR, NEW CASTLE COUNTY														
UNADJUSTED %	58.0%	17.0%	10.0%	6.0%	4.0%	2.0%	1.0%	0.7%	0.6%	0.5%	0.3%	0.1%	0.3%	100.5%
ADJUSTED %	57.7%	16.9%	10.0%	6.0%	4.0%	2.0%	1.0%	0.7%	0.6%	0.5%	0.3%	0.1%	0.3%	100%
INCRMNT. PRECIP.	6.93	6.26	6.17	5.19	4.46	2.73	1.61	1.30	1.27	1.18	0.78	0.29	1.34	39.5
PRECIP. CAPTURE	6.93	6.26	6.17	5.19	4.46	2.73	1.61	1.30	1.19	1.00	0.60	0.20	0.60	38.2
IMPERV. EVENT	0.04	0.19	0.37	0.59	0.82	1.05	1.30	1.54	1.79	2.04	2.29	2.54	4.17	
IMPERV. ANNUAL	2.48	3.13	3.71	3.50	3.25	2.10	1.29	1.07	1.07	1.01	0.68	0.25	1.24	24.8
IMPERV. CAPTURE	2.48	3.13	3.71	3.50	3.25	2.10	1.29	1.07	0.86	0.73	0.45	0.15	0.37	23.1
PERV. EVENT	0.00	0.00	0.00	0.02	0.04	0.08	0.12	0.17	0.23	0.29	0.36	0.44	1.09	
PERV. ANNUAL	0.00	0.00	0.02	0.11	0.17	0.15	0.12	0.12	0.13	0.12	0.08	0.03	0.15	1.2
PERV. CAPTURE	0.00	0.00	0.02	0.11	0.17	0.15	0.12	0.12	0.10	0.09	0.05	0.02	0.04	1.0
50% PERV. ANNUAL	1.24	1.57	1.86	1.81	1.71	1.13	0.70	0.60	0.60	0.57	0.38	0.14	0.69	13.0
50% PERV. CAPTURE	1.24	1.57	1.86	1.81	1.71	1.13	0.70	0.60	0.48	0.41	0.25	0.09	0.21	12.0
CAPTURE PERCENT	PRECIPITATION			96.8%	IMPERVIOUS		93.2%	PERVIOUS		83.2%	50% COMBINED			92.7%
DOVER, KENT COUNTY														
UNADJUSTED %	53.0%	18.0%	11.0%	6.0%	4.0%	3.0%	1.0%	0.9%	0.8%	0.6%	0.2%	0.2%	0.5%	99.2%
ADJUSTED %	53.4%	18.1%	11.1%	6.0%	4.0%	3.0%	1.0%	0.9%	0.8%	0.6%	0.2%	0.2%	0.5%	100%
INCRMNT. PRECIP.	6.41	6.71	6.88	5.26	4.52	4.14	1.63	1.70	1.71	1.43	0.53	0.58	2.27	43.8
PRECIP. CAPTURE	6.41	6.71	6.88	5.26	4.52	4.14	1.63	1.70	1.61	1.21	0.40	0.40	1.01	41.9
IMPERV. EVENT	0.04	0.19	0.37	0.59	0.82	1.05	1.30	1.54	1.79	2.04	2.29	2.54	4.17	
IMPERV. ANNUAL	2.30	3.36	4.13	3.55	3.29	3.19	1.31	1.40	1.44	1.23	0.46	0.51	2.10	28.3
IMPERV. CAPTURE	2.30	3.36	4.13	3.55	3.29	3.19	1.31	1.40	1.16	0.89	0.30	0.31	0.63	25.8
PERV. EVENT	0.00	0.00	0.00	0.02	0.04	0.08	0.12	0.17	0.23	0.29	0.36	0.44	1.09	
PERV. ANNUAL	0.00	0.00	0.02	0.11	0.18	0.23	0.12	0.15	0.18	0.18	0.07	0.09	0.55	1.9
PERV. CAPTURE	0.00	0.00	0.02	0.11	0.18	0.23	0.12	0.15	0.15	0.13	0.05	0.05	0.17	1.4
50% PERV. ANNUAL	1.15	1.68	2.08	1.83	1.73	1.71	0.71	0.78	0.81	0.70	0.27	0.30	1.33	15.1
50% PERV. CAPTURE	1.15	1.68	2.08	1.83	1.73	1.71	0.71	0.78	0.65	0.51	0.18	0.18	0.40	13.6
CAPTURE PERCENT	PRECIPITATION			95.7%	IMPERVIOUS		91.3%	PERVIOUS		71.8%	50% COMBINED			90.1%
GEORGE TOWN, SUSSEX COUNTY														
UNADJUSTED %	56.0%	17.0%	10.0%	6.0%	4.0%	2.0%	1.0%	0.7%	0.5%	0.4%	0.2%	0.2%	0.6%	98.6%
ADJUSTED %	56.8%	17.2%	10.1%	6.1%	4.1%	2.0%	1.0%	0.7%	0.5%	0.4%	0.2%	0.2%	0.6%	100%
INCRMNT. PRECIP.	6.82	6.38	6.29	5.29	4.54	2.78	1.64	1.33	1.08	0.96	0.53	0.58	2.74	41.0
PRECIP. CAPTURE	6.82	6.38	6.29	5.29	4.54	2.78	1.64	1.33	1.01	0.81	0.41	0.41	1.22	38.9
IMPERV. EVENT	0.04	0.19	0.37	0.59	0.82	1.05	1.30	1.54	1.79	2.04	2.29	2.54	4.17	
IMPERV. ANNUAL	2.44	3.19	3.78	3.57	3.31	2.14	1.32	1.10	0.91	0.83	0.46	0.51	2.54	26.1
IMPERV. CAPTURE	2.44	3.19	3.78	3.57	3.31	2.14	1.32	1.10	0.73	0.60	0.30	0.31	0.76	23.5
PERV. EVENT	0.00	0.00	0.00	0.02	0.04	0.08	0.12	0.17	0.23	0.29	0.36	0.44	1.09	
PERV. ANNUAL	0.00	0.00	0.02	0.11	0.18	0.16	0.12	0.12	0.11	0.12	0.07	0.09	0.67	1.8
PERV. CAPTURE	0.00	0.00	0.02	0.11	0.18	0.16	0.12	0.12	0.09	0.09	0.05	0.05	0.20	1.2
50% PERV. ANNUAL	1.22	1.60	1.90	1.84	1.74	1.15	0.72	0.61	0.51	0.47	0.27	0.30	1.60	13.9
50% PERV. CAPTURE	1.22	1.60	1.90	1.84	1.74	1.15	0.72	0.61	0.41	0.34	0.18	0.18	0.48	12.4
CAPTURE PERCENT	PRECIPITATION			95.0%	IMPERVIOUS		90.2%	PERVIOUS		67.0%	50% COMBINED			88.8%

Table 4-1 also displays capture volumes in terms of percentages of annual runoff, instead of annual rainfall. Event runoff is calculated according to the methods described in Chapter 5, using the average rainfall depth for each increment. Runoff volumes are presented for impervious surfaces with a curve number of 98, pervious surfaces at a curve number of 61, and a combined area that is 50% pervious. Annual depth is incremental rainfall multiplied by the proportion of runoff to rainfall at each increment. At rainfall increments greater than the capture depth of 2.0 inches, captured runoff is the product of annual depth times the ratio of capture depth to upper increment of annual rainfall, less 10% to account for the fact that runoff is less than rainfall. (Actual values of this relationship vary from 5% to 16%, depending upon depth.)

Depending upon the rainfall distribution, 2.0 inches of rainfall capture captures 90% to 93% of the annual runoff from impervious surfaces, but only 67% to 83% from the pervious surfaces. However, when impervious areas are 50% of the total, the total runoff capture percentage ranges from 89% to 93%. This is due to the fact that runoff from impervious areas is much greater than that from pervious areas, exceeding pervious runoff volumes by over a factor of 10. For this reason, changes in pervious curve numbers do not affect the total percentages materially. A capture depth of 2.0 inches thus ensures treatment of at least 80% of annual runoff for the typical development site in which impervious surfaces comprise at least 35%. If the capture threshold were a smaller rainfall event, such as 1.0 inch, the annual capture volume from low-density sites would be less than 80 percent.

Given that BMPs are not 100% effective (typically 80-95% for TSS, 40-60% for total phosphorus, and 0-30% for total nitrogen), this implies that a higher percentage of annual rainfall volume should be captured by a BMP to attain target levels of treatment. If the intent is to provide an 80% reduction in annual loads, the capture percentage has to be multiplied by treatment efficiency. At an efficiency of 90%, reductions in TSS loads would be 83%, 80% and 81% for 50 percent impervious sites in New Castle, Sussex, and Kent Counties, respectively. Since these values correspond to an 80% target, this analysis suggests that a treatment volume of at least 1.5 inches is necessary. This corresponds to the findings established in Wisconsin (WDNR, 1995).

BMPs will provide for more treatment and last longer when the treatment volume is thus increased by 100% over the one inch typically used. Furthermore, BMPs properly designed for a volume of 2.0 inches can often route larger flood flows effectively with a relatively small increase in total storage volume. Such BMPs also provide for extensive contact filtering and the opportunity for infiltration of runoff. Therefore, Green Technology recommends that runoff quality BMPs should be designed to capture and treat runoff volumes for storm events up to 2.0 inches of rainfall.

To promote recharge, designers are encouraged to install BMPs that recharge the losses in recharge volumes due to development. Since this can be difficult in certain sites, recharge is not a requirement of this Manual. Wherever recharge is not a viable option due to site conditions, extended detention of the 2.0 inch volume is recommended, as discussed in more detail in Section 4.2.

4.2 CRITERIA FOR STREAM BANK PROTECTION HYDROLOGY

The need to mitigate the changes in hydrology from urbanization is thus particularly important. Regardless of whether pollutants are present, healthy benthic communities cannot exist in unstable streams. Therefore, measures to alter urban hydrologic responses to that which the receiving stream can tolerate are vitally important to the health of streams. The following discussion sets forth the background and rationale for criteria to protect instream habitat.

In typical soils, infiltration of the entire increase in runoff volume is rarely feasible in developments where imperviousness exceeds 20% or so. Therefore, extended detention of the frequent runoff events is the best option available to minimize downstream bank erosion. As a result, Ontario regulations have required Overcontrol (OC) of the one inch storm by retaining this volume over a 24 hour period. However, too much OC by itself can cause aggradation, which also destabilizes streambanks. Instead, MacRae (1991) recommends the Distributed Runoff Control (DRC) approach, in which the retention structure is designed to increase OC volumes to reduce post-development flows by at least 50% below pre-development flows, at stages below midbank flows. The outflow increases to equal pre-development peak flows of the two-year storm at the bankfull flows. This provides a substantial reduction in the erosional potential of sub-bankfull flows, while allowing the flows at the higher stages to flush out accumulated sediments.

Since midbank flows are the dominant flows causing bank erosion, the midbank flow rate is used as the pre-development target criterion for the 50% over-control release rate. Assuming that the two-year recurrence interval event represents bankfull discharge, and that mid to upper bank flows approach 50% of this flow, design flow would occur at precipitation events in the range of 1.4 to 1.7 inches, representing a recurrence frequency of several times per year. Since these values are close to the 2.0 inches required for quality treatment, Green Technology recommends a reduction in the peak rates from pre-development flows from the storm of 2.0 inches of rainfall, and no increase in the 2 year peak runoff rate.

However, the appropriate DRC reductions from pre-developed flows and the criteria for pre-development conditions are more difficult to establish. Booth and Jackson (1997) note that the threshold for impacts from urbanization occurs at a 10% effective impervious area (EIA), or 20% total impervious area (TIA) typical of medium density single family homes. Using the HSPF model, Booth and Jackson also noted that this threshold was equivalent to a Q_{2post} (post-development peak flow rate from the two-year storm) being equal to or less than Q_{10pre} for forested areas. This tolerance value implies that forested streams could convey up to a 60% increase in flows (the difference between Q_2 and Q_{10}) before destabilization occurs. The 10% EIA threshold seems to correlate well with the 10% to 15% TIA threshold for habitat impacts noted for streams in Delaware by Maxted (1997). Ongoing work in Vermont suggests that an EIA of up to 10% is tolerated by cohesive stream banks, but the most sensitive streams could only handle up to 3% EIA before degradation was noted (MacRae, personal comm.). Note that the threshold concept is not entirely accurate since the decline in benthic indices in proportion to urbanization occurs along a continuum; however, it provides a basis for the criteria discussed above (Maxted and Shaver, 1999).

As discussed above, the extent of DRC reduction depends upon the cohesion of the stream bank toe stratigraphic unit. The Plasticity Index (PI) of the streambank toe seems to correlate highly with cohesion, so it is possible that bank toe information can be estimated from NRCS Soils Mapping. However, this provides a very rough estimate at best, and field measurements are needed to verify the degree of bank stability of the receiving stream. Depending upon the relative stability of the toe unit, MacRae (1991) recommends DRC targets up to 90%. Note that each stream is different, and a DRC percentage beyond the optimal is projected to actually increase bank erosion due to aggradation. Therefore, site-specific revisions to these targets may thus be required, as discussed in more detail below. Since the relative bank stability of the streams has not been methodically evaluated in Delaware, the literature supports a conservative approach requiring at least the minimum protection of a 50% DRC.

However, nearly all of the available land in the Piedmont physiographic region in Delaware has been developed already, with severe impacts upon the receiving streams. Therefore, overcontrol of the quality storm from new developments in this area would have minimal benefits on already impacted streams. Only in the few watersheds that remain pristine would such an approach be warranted. The vast majority of development now occurs in the Coastal Plain physiographic region, where stream gradients are low enough that bank erosion from urban runoff is not a problem (Dickey, personal communication).

Therefore, Green Technology recommends that the criteria for stream bank protection in the Coastal Plain require that post-development peak flows of 2.0 inches of rainfall match pre-development peak flows. Likewise, there shall be no increase in the peak rate of the two-year runoff event. Stricter criteria using forest cover with 10% EIA for pre-development conditions and a 50% reduction in the peak flows for the 2.0 inch event may be applicable to pristine Piedmont watersheds. Modeling BMPs for selected sites designed for these storm events using continuous simulation models such as PCSWMM may refine this runoff criterion.

4.3 OFFSITE FACTORS IN STREAM BANK PROTECTION

A key factor in selecting required levels of peak flow control is an examination of the status of bank stability in the area affected by development. In alluvial streams where the floodplains have been subject to aggradation and subsequent incision, nickpoints caused by increased flows occur at locations of bed instability. (Nickpoints are the drop in bed elevations where a normal channel falls into an incised channel.) This channel incision alters equilibrium conditions upstream by increasing the local gradient. This results in a large increase in stream energy, causing channel incision to progressively migrate all the way up through the watershed until equilibrium is reestablished (Booth, 1990). Thus, degradation of the “local base level" by channel incision has a most important and far reaching impact (Heede, 1986).

Harvey and Watson (1986) note that there are several stages of stability in this type of alluvial stream. Stage I corresponds to a relatively stable stream section in the headwaters above a nickpoint. Stage II corresponds to the deeply incised channel downstream of the nickpoint. Stage III comprises stream reaches undergoing peak rates of bank erosion due to incision, where the channel rapidly widens. Stage IV occurs where the channel has begun to stabilize, and new floodplains have become established within the limits of the widened channel. Stage V represents a stream reach that has regained dynamic equilibrium, where the original floodplain is now a relict terrace above the new floodplain.

This sequence in the geomorphology of incised channels suggests that stream bank protection measures would be most successful in Stages IV and V (Harvey and Watson, 1986), while nickpoints should be stabilized to preserve Stage I reaches (Harvey and Watson, 1986; Heede, 1986; Booth, 1990). After farming declined in the earlier part of the 20th century in the Delaware Piedmont, most of the streams had attained Stages IV and V by the 1950s. However, due to a new episode of downcutting and bank erosion from urban development over the last few decades, many of these streams are now in Stages II and III. In many of these cases, even the highest level of onsite runoff control will not be adequate, particularly where runoff hydrology has been irretrievably altered by existing development. In the Coastal Plain, bank erosion impacts due to urbanization have been generally less extreme (Dickey, personal comm.). This is due to the lower gradient, as well as the fact that less aggradation has occurred in the first place, and much of the coastal plain remains in agricultural uses.

A further consideration is the role of streamside vegetation. Where streamside vegetation had been removed, highly elevated temperatures and lowered DO levels were observed in Delaware Coastal Plain streams (Maxted and others, 1995). Whipple and others (1981) noted that riparian vegetation increased bank stability in urbanizing creeks. Shields and others (1995) were able to successfully stabilize noncohesive alluvial stream banks by establishing native vegetation. Vegetation decreased bank erosion by up to 80% along a creek in British Columbia (Beeson and Doyle, 1995). Horner and others (1997) noted relatively healthy metrics of benthic diversity in watersheds up to 30% TIA where streams were well buffered with riparian vegetation. Zucker and White (1996) also noted a high correlation between indices of biological integrity and the extent of riparian buffers. Yoder and Miltner (2000) report that habitat and IBI indices are increased in urban areas with intact riparian zones. Horner and others (2000) recommend a continuous riparian buffer of native vegetation as one the best BMPs for streams impacted by urban runoff.

These findings underlie the current thrust in stream restoration efforts with soil bioengineering (Kondolf and Micheli, 1995). Vegetative stabilization is the basis for the potential of RBS and SBR BMPs to provide protection of eroding stream banks. In coordination with onsite controls, RBS and SBR BMPs can provide substantial protection of stream banks and improve streams when compared to existing conditions.

In watersheds where streams are heavily impaired by widespread development, onsite hydrological controls that assume a relatively pristine original condition would be ineffective for new development, and thus represent a wasted expense. In these circumstances, RBS and SBR efforts and structural BMPs, such as regional stormwater facilities and retrofits of existing facilities, would be the most effective approach for restoration. This underlies the need to formulate River Corridor Management Plans (RCMPs) that take into account the status of existing and anticipated watershed development, the destabilization stage of each reach, the susceptibility of streambanks to erosion, and the extent of intact riparian buffers. The cumulative implications of these factors would be used in the RCMP to formulate the appropriate priority for various BMPs, and their target criteria.

In an extremely impaired watershed, a RCMP may permit onsite peak flow controls to use existing average watershed land cover in establishing pre-development conditions, with cost savings to be applied toward offsite BMPs. On the other hand, development in pristine streams with little or no degradation should be subject to the most stringent onsite control requirements, which could likely exceed the 50% DRC reduction, and require a lower EIA in establishing existing conditions. Moderately impaired watersheds could vary between these extremes, depending on the potential for regional facilities and stream bank stability. Where regional offsite facilities are available or projected, onsite control criteria for bank erosion may be relaxed in exchange for pro-rated contributions toward the regional facilities. Where unavailable, the onsite controls would be applied. Absent modifications explicitly incorporated into a RCMP, the minimum required controls would apply to all projects.

4.4 CRITERIA FOR FLOODING EVENTS

Additional criteria include the 10-year storm for conveyance design, and the 100-year storm for flood control. Excluding discharge to tidal waters or situations where regional BMPs are proposed, existing site conditions (without impervious surfaces) represent the targets for the runoff controls.

As 10-year events are generally cyclonic or frontal storms with long interevent intervals, a longer drawdown time of 48 hours is appropriate for storage routing, while avoiding interference from subsequent events. This is important in reducing the flooding potential of synchronized flow peaks from sites with typical 10-year controls. Given that the midbank flood control structure is designed to release its storage volume over 24 hours, the higher control structure would be designed to release its storage volume over the preceding 24-hour period. It is quite possible that the same orifice design for the 2-year peak flow reductions discussed in Section 4.6 would address this requirement in most cases.

Therefore, Green Technology recommends that the design criteria for the 10-year event require that the peak rate be no greater than the pre-development rates under existing conditions (including existing impervious surfaces), and that this volume be released over as long a period as possible. The design criteria recommended for the 100-year event also requires that the peak rate be no greater than the pre-development rates under existing conditions.

CHAPTER 5 URBAN RUNOFF HYDROLOGY

5.1 URBAN RUNOFF MODELS

The hydrological analysis required for urban runoff BMP design must identify the runoff contributions from the various land cover components of a project, as affected by soil characteristics and land cover type. It must also address how such runoff changes in response to rainfall events of differing intensities and precipitation amounts. The hydraulic design elements must be able to realistically calculate the flow path components of runoff, and route runoff through storage structures. It should also be capable of partitioning overland discharge from subsurface infiltration components.

Continuous simulation models such as PCSWMM and HSPF are acknowledged as the most accurate tools for this purpose. Even though these models are now accessible to the desktop, considerable training is required to use them properly, and their extensive data collection, calibration and verification requirements preclude practicality for design of BMPs at the site level. SLAMM (Pitt, 1987) is a simpler continuous model that provides excellent hydrological results in urban watersheds of interest. However, it aggregates many important input parameters for pervious and impervious areas, and has no routing components.

Now that powerful computers are widely available, event-based models have been deemed outdated, inadequate, and even unethical (James, 1994). Simple design storm models are thus considered inappropriate to address receiving water quality issues (Pitt, 2000). However, event based modeling can provide appropriate results when the parameters are properly calibrated by comparison to continuous modeling (Strecker and Reinaga, 2000). These authors noted that the curve number (*CN*) method overestimates runoff peaks in large storms when antecedent moisture conditions were classified as saturated. Using event-based models based upon Hortonian type infiltration equations with decay coefficients, Guo and Adams (1998) and Nnadi and others (1999) report good results compared to continuous simulation modeling. These authors also noted that the *CN* method tended to overestimate runoff in the latter study, apparently due to the influence of antecedent moisture.

The *CN* method has been well documented as an excellent watershed loss model for flooding events (Woodward and others, 2002a). On the other hand, the *CN* method substantially underestimates runoff from small urban watersheds in the small storm events that comprise the great majority of total annual runoff (Pitt, 1987, 2000). Errors in peak flow measurement in these events can range up to 1350 percent (Fennessey and Hawkins, 2002). Indeed, even the authors of the *CN* method have recognized that the curve number for small events (less than 2 inches) must be much higher than the stabilized curve number in order to obtain the observed results (Van Mullem and others, 2002; Hjelmfelt and others, 2002).

Conversely, the *CN* method substantially overestimates runoff peaks (by up to 1000 per cent) from forested watersheds in small storms, since runoff follows shallow subsurface flow pathways with little overland response (Fennessey and others, 2001; Fennessey and Miller, 2002). Since these errors compound each other, using the *CN* method to design BMPs for small storm events where the undeveloped condition is a wooded watershed would result in detention designs that are quite deficient for the events of interest in Green Technology BMP design. Furthermore, even within the same soil *CN* polygon, there can be substantial variations in *CN* depending upon landscape position, with upland *CN*s being less than half the *CN* in concavities where a saturated zone occurs during storm events (Fennessey and Miller, 2002, Fennessey and Hawkins, 2002).

Notwithstanding these limitations, the *CN* method used in TR-20 still remains the method of choice in the design and regulatory community for designing stormwater management facilities. By disaggregating different combinations of land cover and soil type, TR-20 performs well in the larger flooding events of interest in stormwater quantity management, and addresses many of the factors involved in continuous simulation models. TR-55 was subsequently formulated to simplify TR-20 for smaller watersheds. While reasonably close to TR-20 results (within 5% to 10 %), TR-55 is simpler, and used as a basic hydrology program for relatively small, less complex watersheds.

The Delaware Urban Runoff Management Model, or DURMM, has adapted the *CN* method to address the need for a relatively simple, but more accurate, hydrologic modeling approach for small storm events. DURMM hydrology is based upon the Hortonian infiltration equation, as incorporated into the SLAMM model. Along with the pollutant loading elements discussed in Chapter 3, and the pollutant removal routines discussed in Chapters 7 through 11, DURMM is an integrated model for predicting urban runoff volumes, their pollutant loads and their removal by BMPs.

Since TR-20 is already required in Delaware for the design of structural BMPs, DURMM uses the same allocation of discrete polygons defined by land cover and soil group as is used in the *CN* method. Likewise, the determination of the segmental flow pathways used to determine time of concentration (*Tc*) is similar to that used in TR-20. However, DURMM runoff volume computations follow the SLAMM hydrologic equations for small storm event runoff volumes, and they employ a flow-based approach to routing the segmental flow pathways. In this manner, the precision of TR-20 input procedures is complemented by the greater accuracy of the SLAMM computational algorithms and flow-responsive *Tc* computations.

5.2 DURMM MODEL

TR-20 calculates runoff according to the following equation from NRCS (1985):

$$Q = \frac{(P - Ia)^2}{(P - Ia) + S} \tag{1}$$

where Q is runoff volume, P is precipitation, S is storage (a term roughly equivalent to accumulated infiltration losses) and Ia is initial abstraction, the initial losses due to interception, depression storage, and evaporation. All units are in terms of depth of runoff and precipitation.

In english units, storage (inches) is related to Curve Number (*CN*) according to the following equation:

$$S = \frac{1000}{CN} - 10 \tag{2}$$

In TR-20, *Ia* is fixed at 0.2*S*, so runoff relates to precipitation and storage as follows:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \tag{3}$$

At a given *P*, as *CN* decreases, *S* increases, so *Q* decreases. The more “pervious” the soil cover complex, the greater the storage, which is rarely exceeded in the most pervious areas. As *P* declines toward 0.2*S, Q* approaches 0, so no runoff occurs from pervious areas in small rainfall events.

Note also that, while *Ia* is a function of *S*, its effect is to eliminate initial precipitation, even before any *S* is absorbed. The value of *S* is thus independent of *P*, and assumes that no further storage will occur once *P* exceeds *S*. In essence, the soil group/land cover complex is treated as a sponge with a fixed capacity that begins to fill up once *P* exceeds *I*a.

This approach is not at all the behavior of a field scale infiltration model, where infiltration rates will increase with increasing precipitation intensity (Pitt, 1987; Woodward and others, 2002a). Instead, the *CN* method integrates many components of the entire watershed response, including the partial contributing area (PCA) concept (Van Mullem and others, 2002). PCAs are relatively small saturated areas in downslope concavities adjacent to streams, which can be responsible for much of the runoff generation (Kirkby, 1988; Pionke and others, 1988; Gburek, 1900). PCAs are linked to upslope contributory areas by largely subsurface flow pathways during the rainfall event (Pionke and others, 1988). These subsurface flow pathways also directly contribute to the runoff response (NRCS 1985, Hjelmfelt and others, 2002).

As such, the *CN* method was intended to model the entire watershed response, not that of saturation excess flows (which occur very rarely) from each individual contributory land cover complex polygon in upland areas (Van Mullem and others, 2002). Therefore, its use at the small site scale must be viewed cautiously, as noted by Fennessey and associates (Fennessey and Miller, 2002; Fennessey and Hawkins, 2002). Nonetheless, its rainfall/runoff response is considerably more realistic than the Rational Method, and it is already very familiar to the engineering and regulatory community. As such, the *CN* method provides the best available starting point for developing an approach to designing urban runoff BMPs. By providing the modifications discussed below, DURMM attempts to rectify many of the shortcomings of the *CN* approach.

In the agricultural watersheds and larger storm events for which TR-20 was formulated, the value of *Ia* does not substantially affect results, since *P* is usually much greater than 0.20*S*. This value was chosen to best fit the observed data from many watersheds (NRCS, 1985) as a mean between 0.00 and 0.30 (Ponce and Hawkins, 1996). However, for predicting the hydrology of small urban watersheds under smaller storm events, this fixed value for *Ia* in TR-20 has serious shortcomings, and thus TR-20 tends to grossly under predict the hydrological contribution during smaller rainfall events from the lawns and landscaping that comprise urban pervious areas (Pitt, 1987, 2000). This is part of the reason why the curve number in smaller events must be increased to match observed responses (Van Mullem and others, 2002). Indeed, a recent paper by the NRCS researchers suggests that the initial abstraction should closer to 0.05*S*, not to the 0.20*S* originally selected in TR-20 (Woodward and others, 2002b).

For these reasons, equation (3) is only used for runoff events of large magnitude over 5.0 inches (125mm) in DURMM. For the smaller events used in the design of Green Technology BMPs, a different relationship is required to obtain more accurate results.

From extensive comparison of the *CN* method against direct observations of urban runoff events, Pitt (1987, page 199) suggests the following relationship for runoff as a function of *P* (in mm) and *g*, a coefficient based upon *CN*:

$$Q = P - S'(1 - e^{-gP}) \tag{4}$$

where *S'* is accumulated losses (in mm) after initial abstraction, or *S* - *Ia*. Note that (4) is very similar to the original equation developed by Victor Mockus in formulating the *CN* method (see Mishra and Singh, 1999).

Pitt (1987) provides a table of *g* coefficients regressed to *CN*. The following equation and coefficients were developed to match the values in Pitt's table as a function of *CN*:

$$g = Ae^{B(CN)} + \frac{Ce^{D(CN-B)}}{10{,}000{,}000} \tag{5}$$

A	B	C	D	E
0.00065	0.0364	155	0.49	80

The first term in (5) is formulated to address the lower *CN*s of pervious areas, while the second term addresses the more direct runoff response of impervious *CN*s. Using these coefficients, equation (5) reproduces the regressed values for coefficient *g* tabulated by Pitt (1987) within 11%.

To get the best fit between (4) and Pitt's (1987) observations for pervious areas, *S'* is set at -0.075, While this negative *Ia* is counterintuitive, it provides the best match to the observations shown in Figure 5-1. Many think *Ia* should be closer to 0.05 for urban areas; for instance see Woodward and others (2002b) who suggest that this value is more appropriate for even rural areas. However, while an *Ia* of 0.05*S* gives a better fit to the observations than 0.20*S*, it is not as good a fit as the negative value used. See also the results presented in Mishra and Singh (1999), in which their model with an *Ia* 0 gave the best fit to observed data for a wide variety of watersheds.

Therefore, there is precedent for the assumption that *S'* would approach *S* for the urban sites of interest in DURMM. When the *CN* is adjusted to correspond to TR-20 values at 125 mm rainfall depths, the value of *S'* used matches observations of runoff from pervious surfaces, as shown in Figure 5-1. The *CN*s allocated for the pervious areas are quite close to the *CN*s used in

TR-20 for lawns in good condition. The curves thus match the observations of the data set as closely as possible, and blend into the TR-20 curves at rainfall depths of 125mm (5 inches).

As shown in Figure 5-1, this curve fit method projects runoff volumes as a function of *CN* that are quite close to runoff observed by Pitt (2000). Note how the runoff volumes at low rainfall depths are much greater than those projected by the *CN* method, while runoff volumes at higher rainfall depths are less than the *CN* method.

This trend in rainfall/runoff relationships follows the observations of rainfall/runoff relationships and declining *CN* values as rainfall amounts increase reported by Pitt (1987), Fennessey and Hawkins (2002), Van Mullem and others (2002), and Mishra and Singh (1999).

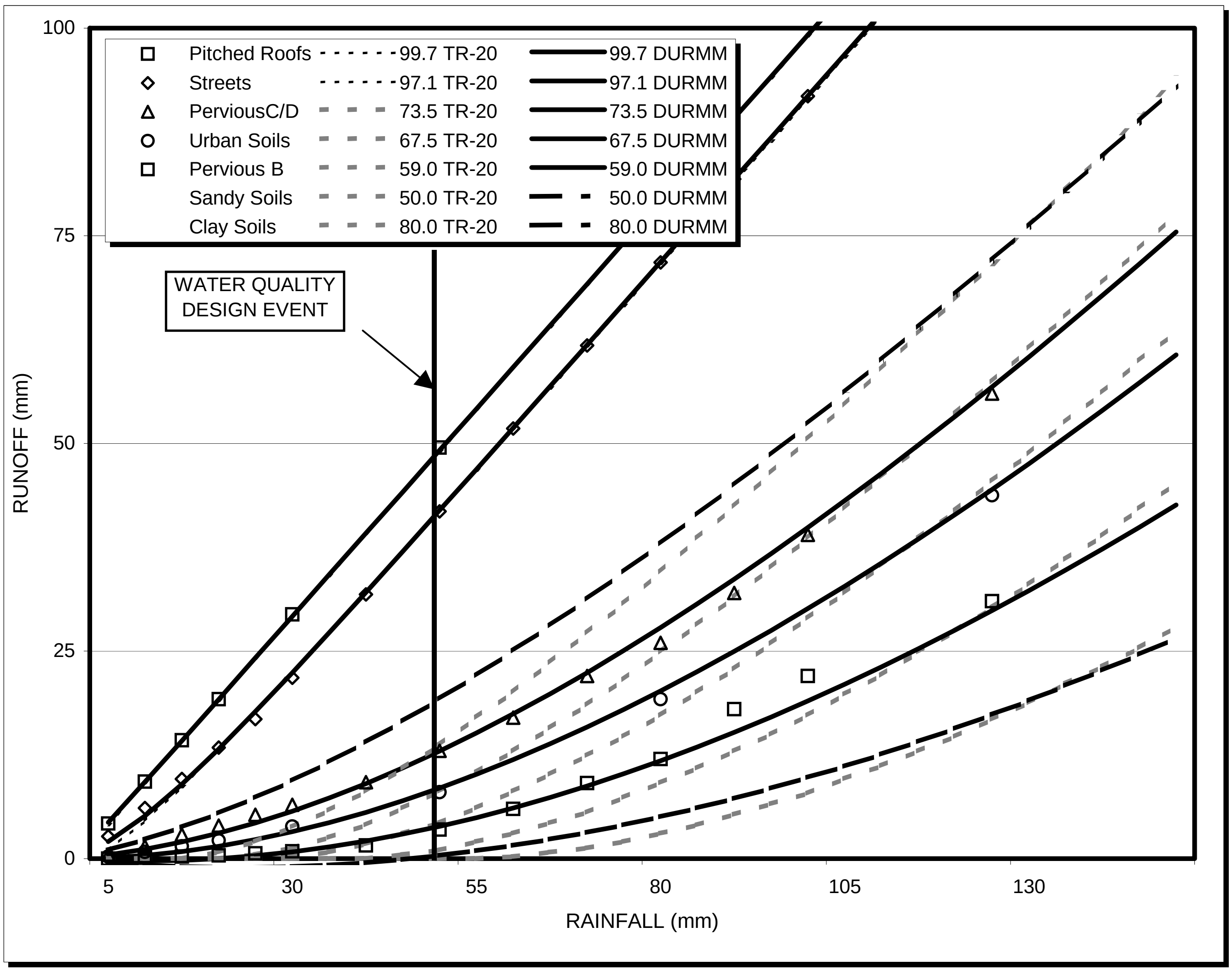

Figure 5-1: TR-20 Runoff, DURMM Runoff, and Observations from Pitt (1987)

5.3 RUNOFF VOLUMES- STORM EVENTS

The runoff predicted from small storms on urban sites using (4) is thus better related to *CN* and *P* than in TR-20. Accuracy in this relationship is essential to realistically model the response of urbanizing area to small precipitation events. At rainfall depths below 60 mm, note how TR-20 runoff volumes (in dotted lines) are much less, or nonexistent, when compared to the curves generated by DURMM routines. Instead, the DURMM curves pass close to the values observed by Pitt at rainfall depths between 25 and 80 mm, the design rainfall amounts for quality and streambank protection routing. To verify that DURMM curves extend over the applicable range of *CN*s for pervious areas, curves are included for sandy soils and clay soils with *CN* values of 50 and 80, respectively. This trend in rainfall/runoff responses follows that generated by reducing *Ia* from 0.20*S* to 0.05*S* as reported by Woodward and others (2002b).

Note the form of equation used in DURMM has the natural log relation used in Hortonian infiltration, and it does not limit infiltration losses as TR-20 does (where *S* is a fixed value). This results in DURMM curves falling below the TR-20 values at higher rainfall depths, as noted by Pitt (1987), Mishra and Singh (1999), and Woodward and others (2002b). This suggests that infiltration rates may increase (and therefore *S* is not a constant value) during more intense rainfall events, as noted by Pitt (1987, 2000).

Taking a conservative approach, DURMM uses the *CN* method equations for the conveyance and flooding events at rainfall depths over 125 mm (5 inches), where the curves between the two methods were selected to meet. Although the crossover threshold in a very pervious watershed occurred at a higher value of 300 mm (Mishra and Singh, 1999), the response of this arid watershed is not applicable to watersheds in the humid east. The effective difference between DURMM and TR-20 is minor at high values of *P*, especially considering that BMP designs evaluate the difference between predevelopment and postdevelopment conditions.

Another very important point raised by Pitt (1987) is that impervious runoff *CN*s are by no means exactly 98, as assumed in TR-20. Impervious surfaces have a certain amount of infiltration and depression storage, depending upon surface roughness and slope, particularly in the case of roofs. Pitched roofs or extensive paving such as parking lots (which permit little infiltration at the edges) have a high *CN* of 99.7, while flat roofs have a *CN* of 98.1, and narrower streets with average roughness have a *CN* of 97.1. Smooth streets have a higher *CN*, and old rough streets have a lower *CN*. When calculating the relative contributions of runoff from these differing types of impervious surfaces, the fine differences between these values are important at low rainfall depths, as shown in Figure 5-1. By eliminating the rounding error introduced in typical TR-20 software packages, DURMM is able to account for these differences. Pitt's values for the differing *CN*s of impervious surfaces are used in the model.

The proper allocation of *CNs* for pervious urban areas is still a matter of some uncertainty. Pitt (1987, 2000) notes that infiltration rates decline over event duration, but increase with rainfall intensity. Compaction has the greatest effect in sandy soils, which would have high rates under natural conditions (Pitt, 2000; Pitt and others, 1999). Clay soils were equally affected by compaction and antecedent moisture. High traffic areas such as playing fields were observed to have infiltration rates even lower than impervious surfaces (Pitt, 1987; 2000; Pitt and others, 1999). A study of lawns on compacted sandy soils in the New Jersey Coastal Plain showed infiltration rates were greatly reduced once bulk density exceeded 1.5

mg/cm^3 (OCSCD, 2001). Originally classified as Hydrologic Soil Group (HSG) A in the uncompacted state, HSGs of the same soils were effectively reduced to C or even D as a result of excessive compaction. The cover condition of these soils would be rated as poor, since plants could not establish an adequate root system.

In contrast to these compaction results, Barros and others (1999) reported that, after a period of adaptation, compacted soils had runoff rates similar to that found under natural conditions. A study of lawn runoff in Wisconsin by Legg and others (1996) noted that there was no runoff from rainfall amounts less than 1 cm, and that rainfall intensity had little effect on infiltration rates. There was substantial variation in runoff rates within individual lawns, and from lawn to lawn. The age of establishment was observed to be a significant variable in determining runoff volumes, with the oldest lawns having the least runoff volumes. Such variations in pervious runoff coefficients were also observed in a study of lawns in Pennsylvania, where infiltration rates were lowest in lawns recently established on compacted areas. Infiltration rates increased as a function of increasing lawn age and soil profile condition, structure and decreasing compaction history (Hamilton and Waddington, 1999).

Well-developed soils with macropores established over time would have better infiltration rates than newly graded sites. This is supported by the findings of Barros and others (1999) that compacted soils returned to a native condition after a period of adaptation over the 9 month time frame of the experimental design. At the end of this period, soils compacted to 90% proctor (nearly suitable for roads), were found to have well developed macropores due to infiltration of applied rainfall. In studies of turfgrass in Wisconsin, Kussow (1994, 1995) reported that runoff volumes from lawns constructed under compaction conditions typically generated by new construction was very low. Appreciable runoff was observed only during the winter when the lawns were frozen.

Recent efforts have examined amending compacted soils by incorporating compost. Pitt and others (1999) note that incorporating compost into the top 10 inches of compacted soils greatly increased infiltration rates, particularly in newer lawns. Runoff responses were commensurate with the increase in infiltration rate. Efforts to improve detention basin performance in New Jersey Coastal Plain using compost have shown substantial increases in infiltration rates (C. Smith, pers. comm.).

Since the underlying design assumptions are directed toward established conditions, the infiltration rates suggested by the older established lawns would be applied in DURMM. Therefore, DURMM presumes that the *CN* values used in TR-20 are applicable to lawns, and that the cover condition is classified as good for typical suburban lawns. For playing fields and urban areas with a high degree of permanent ongoing compaction, the cover condition should be reduced from good to fair to account for compaction that persists over time. In cases of extreme compaction, the cover condition should be classified as poor.

5.4 RUNOFF VOLUMES- ANNUAL DISTRIBUTION

Another approach to evaluate the differing relationships between rainfall and runoff for small events is to compare results from DURMM against that predicted from the *CN* method over the course of a year. Using the annual rainfall distribution set forth in Table 4-1, one can calculate the runoff volume from a given soil cover complex for each quarter inch increment. Multiplied by the percent that each increment comprises of the total rainfall, annual runoff is then derived as the sum of the incremental runoff volumes.

This approach was applied at the landscape scale to the Noxontown Pond watershed in the Delaware Coastal Plain. With an upland area of 5,689 acres, this watershed is largely agricultural, with 62 percent crops, 20 percent forest, 14 percent grass, and an impervious cover of 2.4 percent. Given this setting, pervious runoff, largely from crops, dominates the hydrologic response from the uplands. Noxontown Pond comprises nearly 4 percent of the watershed area, so there is also a substantial volume of direct runoff from the surface of this impoundment.

Upland land cover complex curve numbers were applied from *CN* method, using the values set forth in Table 5-1.

Table 5-1: Land Cover/Soil Group Classifications in Noxontown Pond

NEWTYPE	COVER_	CONDITION_	CURVE NO_A	CURVE NO_B	CURVE NO_C	CURVE NO_D
Agriculture: Cropland	Row Crops, Residue	good	64	75	82	85
Forest: Brush/Shrub	Woods-grass	good	30	58	72	79
Forest: Deciduous	Woods	good	30	55	70	77
Forest: Nursery	Woods-grass	fair	43	65	76	82
Grass: Mixed	Open	fair	49	69	79	84
Grass: Suburban	Open	good	39	61	74	80
Grass: Urban	Open	poor	68	79	86	89
Pastures: Feedlots	Pasture	poor	68	79	86	89
Pastures: Mixed	Pasture	fair	49	69	79	84
Pastures: Open	Pasture	good	39	61	74	80

A GIS was used to determine the total areas of each combination of soil group and land cover, so as to develop the weighted curve number for each land cover type. Note that the cover and condition classification of the crops is rated as good, since the farmers in this area use no-till and cover crops extensively (NCCWRA, 1986). Note also that the grass cover conditions are allocated according to their estimated compaction.

The proportion of rainfall for each quarter inch increment was then applied to the weighted *CN* of each cover type to develop the runoff depth for each rainfall increment. This runoff depth was then multiplied by the percent of rainfall in the increment. Increments were then summed by type to obtain annual runoff depths. Annual runoff depth for each cover type was then multiplied by its surface area to obtain annual runoff volume for each type. Volumes from all types were then summed to obtain the total upland runoff, which was divided by upland area to obtain annual runoff depth.

Using TR-20 routines, the calculated annual runoff depth was only 70 mm (2.77 inches), a value only 7 percent of the annual rainfall of 1,046 mm (41.2 inches). Even applying the worst possible classification to crops (straight row, no residue, poor condition), the runoff depth increased to only 105 mm (4.13 inches). On the other hand, annual runoff using DURMM routines was nearly three times that of the *CN* method, with an annual depth of 189 mm (7.45 inches). Adding the area-normalized runoff depth of 38 mm (1.52 inches) from the pond, the total annual runoff using DURMM routines was computed at 228 mm (8.97 inches).

The USGS has maintained a gauging station for several years on this watershed, so it is possible to obtain a site-specific verification of annual runoff volume. The baseflow component of the runoff hydrographs was determined with the 5 day smoothed minima technique. This is considered the most appropriate method to segregate base flow from runoff (Jordan and others, 1997), particularly in this case, where an impoundment extends stormflow responses. Separation of four years of runoff data resulted in an average annual stormflow volume of 217 mm (8.55 inches). Even though this period is fairly short and rainfall distribution was quite variable, the average rainfall of 1,046 mm is very close to the long-term average of 1,019 mm. This suggests that the value derived from this hydrograph separation is representative of long-term average runoff.

After subtracting direct runoff from the pond, upland runoff by hydrograph separation is computed to be 179 mm (7.05 inches). This value is within 6 percent of that computed by DURMM routines. On the other hand, the *CN* method underestimates this value by over 60 percent. This analysis provides robust support that the routines used in DURMM better replicate actual conditions than the *CN* method for small rainfall events that dominate the annual response.

5.5 IMPERVIOUS DISCONNECTION

Another important factor in determining urban site runoff volumes is the large difference in runoff volumes between pervious and impervious surfaces at a given rainfall depth, as noted in Figure 5-1. Many design manuals and proprietary TR-20 software packages permit averaging the *CN* over both the pervious and impervious surfaces, unless routed as separate subareas during design. However, this approach can lead to substantial errors in runoff volumes, especially during small rainfall events (Panuska and Schilling, 1993; Tsihrintzis and Hamid 1997; Grove and others, 1998).

This simplification has further exacerbated the shortcomings of TR-20 for small storm hydrology. In contrast, DURMM not only has better algorithms for calculating pervious and impervious runoff as a function of rainfall, it calculates runoff volumes from pervious and impervious surfaces as discrete subareas.

It has been recognized for some time that there is a substantial difference in the volume of runoff from impervious source areas that depends upon whether it is conveyed by impervious flow paths (curb and gutters, pipes) versus pervious flow paths (grassed swales). Source areas from which runoff is conveyed by impervious flow paths are defined as connected impervious areas (CIA), as opposed to total impervious area (TIA). Impervious source areas from which runoff is conveyed by pervious flow paths are called disconnected impervious areas (DIA).

A recent detailed study of urban runoff source areas by Lee and Heaney (2003) documented that CIA runoff comprised 72 percent of total annual runoff, even though it was only 36 percent of TIA, and 13 percent of the watershed. The remaining 87 percent of the watershed contributed runoff only when event rainfall exceeded 20 mm. Indeed, the authors of this study noted that the Rational Method was a reliable indicator of runoff volumes in small events only when restricted to the CIA. These authors thus recommended that CIA runoff be explicitly recognized in computing runoff from urban areas.

In contrast to CIA, substantial reductions in runoff volume from DIA source areas have been noted in many studies of the efficiency of filter strips and swales, where pollutant removal efficiencies have been directly correlated with the reductions in runoff volumes (Wanielesta and Yousef, 1993, and many others). This effect of impervious area disconnection has been noted as an important component of Low Impact Development, and has been incorporated into several BMP Manuals, such as the recent Manuals by New Jersey (NJDEP, 2003) and Maryland (MDE, 2000).

By explicitly segregating impervious from pervious areas, DURMM is capable of quantifying the effects of disconnection. To compute the effects of impervious area disconnection, runoff from disconnected impervious areas is allocated as excess precipitation onto the receiving pervious surfaces (Wanielesta and others, 1997, NJDEP, 2003). By entering the area and *CN* of impervious surfaces, and entering the wetted area and *CN* of receiving pervious surfaces, DURMM then calculates the excess precipitation onto, and the resultant runoff volumes from, the receiving wetted pervious surfaces. While runoff from these pervious surfaces will be higher than that from rainfall alone, it can be substantially less than the sum of runoff from the pervious and impervious areas without disconnection.

To account for these effects, DURMM segregates a site into pervious and impervious subareas. The pervious subarea is further subdivided into a natural pervious area, and a graded pervious subarea. The natural area is considered completely undisturbed, so its soil profile and structure must remain intact. As an incentive to leave as much undisturbed area on a site as possible, runoff from natural areas under post-development conditions is computed by TR-20 routines, thus reducing its runoff compared to DURMM routines for small events. While this may not be technically correct for agricultural areas (see Section 5.4 above), it more closely corresponds to the runoff generated from woodlands and abandoned fields (see Fennessey and Miller, 2002). The graded pervious areas use DURMM routines to compute runoff.

Impervious areas are also further segregated into a disconnected impervious subarea, which discharges into the graded pervious area, and a connected impervious subarea that discharges directly into the BMPs via pipes or curb flow. Within the graded pervious subarea, DURMM computes the reduction in runoff volumes from disconnection routines. In this area, the wetted areas would include some of the lawns below downspouts, filter strips below parking lots and the wetted perimeter of swales. Disconnection routines are also applied to the wetted areas of the BMP(s), further reducing the total runoff during the quality event.

DURMM thus explicitly accounts for the reductions in runoff volumes provided by these wetted pervious surfaces. Designs that promote disconnection can reduce runoff volumes for even highly impervious sites by over 50% in the quality event. If the receiving pervious surfaces are very permeable, disconnection can reduce runoff during the quality event by even more. This feature provides a quantitative process-based approach to project the effects of disconnection, which had hitherto been lacking in the literature, let alone in any quantitative design approach.

As the design basis for the impervious area disconnection BMP, disconnection provides the designer with a powerful tool to quantify the benefits of integrating site planning with drainage design. Measures to decrease *CN* by minimizing impervious areas, maximizing natural pervious areas, and increasing afforestation further reduce runoff volumes. In this manner, DURMM provides a powerful tool to quantify the benefits of conservation design, the most fundamental nonstructural BMP.

By using overland conveyance of runoff wherever possible, not only does overland flow reduce volumes (and peak flow rates, as discussed below), it also permits explicit methods for designing BMPs by disconnecting impervious areas and promoting infiltration as an integral part of the design process. Furthermore, there is substantial potential for removal of pollutants when runoff is conveyed through properly designed swales and filter strips. A critical issue is to properly ensure that the receiving pervious surfaces are truly wetted by locating them below flow spreading structures. Otherwise, the wetted area must be constrained to the channel bottom where flows are concentrated.

After accounting for disconnection, the resulting runoff volume is used for generating event hydrographs using TR-55 routines, as discussed below. Once runoff volumes have been calculated for the different source area categories, DURMM computes pollutant mass loads according to the area weighted Event Mean Concentrations (EMCs) of the contributing source areas, as discussed in the DURMM User's Manual.

5.6 TIME OF CONCENTRATION

Time of concentration (*Tc*) determines the peak flow rate for a given runoff volume. *Tc* is usually determined by the segmental method, in which runoff is first conveyed by sheet flow, then by shallow concentrated overland flow, then by channel flow in pipes or streams. In typical usage on small urban sites, these flow paths times decrease according in that order, with sheet flow being the dominant time element, and channel flow usually being very short.

Even though the watershed response normally comprises subsurface flow and runoff from partial contributing areas that are not subject to segmental flow paths (NRCS, 1985), the segmental method is widely utilized. For small urban sites, it seems to be most appropriate, since the other components of runoff are less applicable. On the other hand, under forested pre-development conditions, subsurface flow paths and PCAs often dominate to the runoff response, so the lag method in NRCS (1985) is considered more appropriate (Fennessey and others, 2001).

However, an investigation of the lag equation applied to 1100 events by Folmar and Miller (2000) indicated that the lag equation under-predicts lag in small watersheds, and over-predicts lag in large watersheds, suggesting that lag time varies less as a function of size. Better results could be obtained adjusting the coefficients, and by accounting for the relief and drainage pattern of the watershed. A comparison of the lag method to observed watershed responses for over 50,000 events by Simas and Hawkins (2002) also suggests that the lag equation in NRCS (1985) should be substantially revised. Even so, when revised to better fit the data, the scatter in the watershed responses was extensive.

For these reasons, the approach in DURMM is to use the segmental approach. However, while *Tc* determines the peak rate, the conveyance velocities that determine *Tc* are themselves determined by peak runoff rate. More complex interactions between flow depth, channel shape and vegetative retardance further affect flow velocities, as will be discussed in detail below. However, the TR-55 equation requires a default coefficient to compute shallow concentrated flow conveyance velocities, in which slope is the only variable.

However, comparison of the segmental method to observations by Folmar and Miller (2000) showed that the segmental approach using the TR-55 coefficient for unpaved shallow concentrated flow grossly underestimates the actual lag time (or *Tc*). This is not surprising, since the default *Kv* coefficient of 16.1 used to compute unpaved flow velocities in TR-20 is 80 percent of that applied to paved surfaces. This results in computed flow velocities that are much faster than actually occur in unpaved areas. Since the flow velocities in shallow vegetated swales are typically much less, this would substantially increase travel times closer to that observed.

Sheet flow is defined as unconcentrated flow occurring for the first 100 to 150 feet of the flow path. Topographic features that would concentrate flows define the end of sheet flow. Sheet flow is calculated according to the equation from TR-20 as follows:

$$Tc = \frac{0.007(nL)^{0.8}}{P^{0.5}s^{0.4}} \tag{6}$$

where *n* is Manning's Roughness coefficient, *L* is flow length (feet), and *s* is slope. Note that *P* (inches) in (6) is not restricted to the two-year event, as is used in TR-20. Instead, sheet flow *Tc* decreases as *P* increases. Analysis of the kinematic wave equation that is the basis for (6) shows that as *P* increases, flow velocity increases, and so *Tc* decreases.

DURMM provides for two consecutive segments of sheet flow to model complex flow paths. When flow velocity from this equation is calculated with the Manning's *n* value of 0.24 allocated by TR-20 for dense grass, the resulting velocity is greater than that calculated for swale flows under similar slopes and volumes, as discussed in Chapter 10. This analysis thus suggests that the proper value for turf would be around 0.45, as recommended by Engman (1986) from experimental observations of sheet flow. This value is close to the 0.41 used in TR-20 for BermudaGrass, which has a similar blade density to dense turf of suburban lawns. While this issue deserves further investigation, a Manning's *n* value of 0.24 is allocated to dense turf in the model so as to correspond with the value presently required in Delaware.

At a 2-year rainfall of 3.2 inches, a lawn with a slope of 2 percent for 150 feet would have sheet flow time of nearly 20 minutes. However, in urban sites, the volume of runoff from impervious surfaces in small events is much greater than that from the pervious surfaces. As such, it is thus inappropriate to use pervious sheet flow paths to determine the *Tc*, and an impervious flow path (typically down the most distant driveway) is more realistic. This greatly reduces sheet flow time.

For shallow concentrated flow (or swale flow), DURMM simultaneously solves for peak flows as a function of *Tc*, while solving for *Tc* as a function of peak flows, as affected by the conveyance parameters. TR-55 is used to estimate peak flows, based upon *Tc* and runoff volumes. The equation used in TR-55 for peak flow is:

$$Q_P = q_u AQ / 640 \tag{7}$$

where Q_P is peak discharge (cfs) , Q is the runoff depth (inches), A is area (acres), and q_u is the unit peak discharge (cfs/mi.^{2}). q_u is defined as follows:

$$\log(q_u) = C_0 + C_1 \log(T_c) + C_2 \log(T_c^2) \tag{8}$$

with coefficients C_0, C_1, and C_2 defined as a function of Ia/P according to the table for Exhibit 4 in Appendix F of TR-55 (NRCS, 1986). Given S from the runoff volumes, DURMM calculates Ia/P to the nearest hundredth for linear interpolation between the tabular values to obtain the values of coefficients C_0, C_1, and C_2. Substituted into (8), this gives the peak flow for a given *Tc*.

As discussed in Chapter 6, DURMM provides routines that calculate the velocity of swale flow as a function of conveyance channel geometry, vegetative type and flow depth. Using iterative runs at differing flow depths, swale conveyance design is analyzed to develop total discharge and velocity, which provides the total *Tc* when added to sheet flow *Tc*. Flow depths are adjusted until the flow velocity used to generate swale discharge and *Tc* matches the discharge rate computed by TR-55 using the *Tc* derived from the swale flow calculations.

As in the case of sheet flow, this approach results in decreasing *Tc* as *P* increases. Although this relationship was not observed by Simas and Hawkins (2002) in their analysis of many events in large watersheds, there are many components to the runoff response that are not modeled by segmental flow paths, as discussed above. Such components are largely absent in the small urban sites, where well-documented hydraulic principles of channel flow would determine travel time.

As a result of the swale routines in DURMM, the relative contribution of swale flow to total *Tc* becomes greater than that of sheet flow. This approach reduces the typical dominance of *Tc* by often over-estimated, thus unreliable, and generally inapplicable, sheet flow computations. The swale design parameters can also address channel flow as a narrow, deep swale with a smooth surface cover. By generating a different *Tc* for each storm event, as suggested by Guo and Adams (1999b), DURMM thus generates the *Tc* needed for hydrograph generation of peak flows for the bank protection, conveyance and flooding events.

CHAPTER 6 URBAN RUNOFF HYDRAULICS

6.1 OVERLAND FLOW CONVEYANCE

The preceding discussion emphasizes the importance of swale flow parameters in determining *Tc*, and the resulting peak flow rates. Therefore, a key element in the design of nonstructural BMPs is the design of overland conveyance to retard runoff velocities wherever possible. Not only can overland conveyance slow down flow velocities and thus increase *Tc*, it also disconnects impervious areas and provides for infiltration as an integral part of the design process. Furthermore, the literature indicates the potential for substantial removal of TSS when runoff is conveyed through properly designed swales (Horner, 1988; SWPCD, 1992; Wanielesta and Yousef, 1993).

To address this issue, the methods used in DURMM incorporate detailed swale design features as a fundamental part of the model. The methods set forth in "Biofiltration Systems for Storm Runoff Quality Control" by Horner (1988) are complemented by the work of Barfield and associates (Kao and Barfield, 1978) to form the basis of the following discussion. Modifications are proposed that assist in simplifying the design procedure. By using DURMM routines to relate the variables involved, the accuracy of the design process is also refined. The following discussion reviews the hydraulic elements involved in overland flow through swales, which are defined as wide shallow channels where flows are often submerged below the vegetation height.

6.2 SWALE CROSS-SECTION AREA

Under field conditions, a trapezoidal swale cross-section set forth in the plans evolves over time into a parabolic section at the lower depths involved in biofiltration. For depths below 0.5' (generally well above the maximum depth for filtering), DURMM incorporates a function relating top width *w* to depth *d* for a parabolic swale section. Establishing cross-section shape as a function of depth, it is possible to determine cross-sectional area as a function of depth, and substitute this function for the area term in Manning's equation when determining velocity and flow.

The following expression establishes the half-section profile of the swale at depth *d* as a power function of width *w*:

$$d = A(w/2)^B \tag{9}$$

Rearranging to solve for coefficient A at *d* = 0.5 feet,

$$A = 0.5/(w/2)^B \tag{10}$$

Differentiating (9) (where *d* equals *y*, and *w*/2 equals *x*),

$$dy/dx = AB(w/2)^{B-1} \tag{11}$$

Substituting (10) into (11) and rearranging to solve for *B*,

$$B = dy/dx(w) \tag{12}$$

Since slope at a depth of 0.5 feet is the design side slope *ss* (hor./ver.) of the trapezoidal section, it follows that $dy/dx = 1/ss$, and $w = w_{bottom} + (1.0 \times ss)$, where w_{bottom} is the trapezoidal swale bottom width. Given *B*, *A* is then solved from (10).

Integrating (9) gives the area under the half section as follows:

$$A_{under1/2} = \int A(w/2)^{B} = A(w/2)^{B+1} / (B+1) \quad (13)$$

To obtain swale area, this must be subtracted from the total half area:

$$A_{total1/2} = w/2 \times d = A(w/2)^{B+1} \quad (14)$$

Subtracting (13) from (14), the area of the swale half section is:

$$A_{swale1/2} = A(w/2)^{B+1} \times [1 - 1/(B+1)] = A(w/2)^{B+1} B/(B+1) \quad (15)$$

Multiplying (9) by *w*/2,

$$A(w/2)^{B+1} = wd/2 \quad (16)$$

Substituting (16) into (15), and multiplying by two for the total swale area,

$$A_{swale} = wdB/(B+1) \quad (17)$$

Rearranging (10),

$$w = 2(d/A)^{1/B} \quad (18)$$

Substituting (18) for *w* in (17) gives the cross-section area of the swale section:

$$A_{swale} = 2B/(B+1) \times A^{-1/B} \times d^{(1+1/B)} \quad (19)$$

With coefficients *A* and *B* solved for the design section, it is possible to solve for area as a function of the one variable *d*.

Figure 6-1 shows how the design swale section compares to the typical parabolic swale and trapezoidal swale sections, based upon a 12 foot bottom width and 4:1 side slopes. Note how the swale section remains close to the trapezoidal section at depths up to 1.0 feet, typically the maximum depth under flooding conditions. The trapezoidal section parameters are used to set the construction layout of the swale, which ends up being very close to the design section in the field.

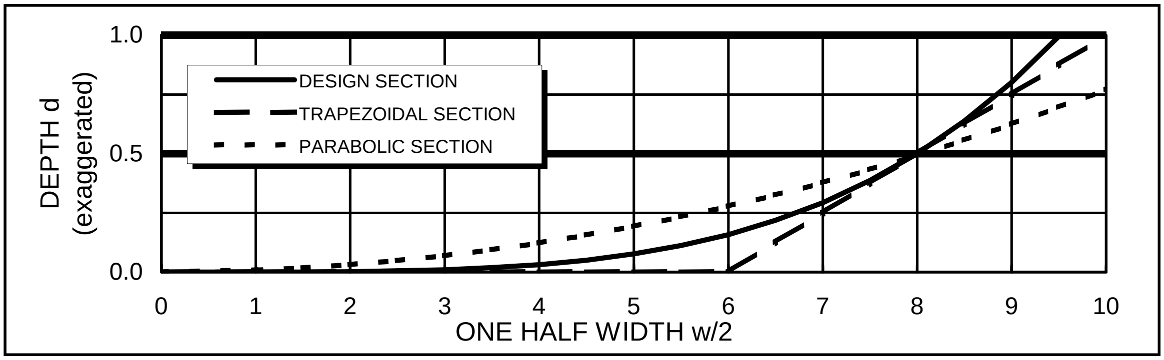

Figure 6-1: Sections for Design, Parabolic and Trapezoidal Cross-Sections

6.3 SWALE HYDRAULIC RADIUS

The wetted perimeter of the swale design function is very close to the width *w* at depths typical for biofiltration. In a trapezoidal section with w_{bottom} of 8 feet and *ss* of 5:1, the difference is less than 1% for depths below 9 inches. This difference would be even less for the design section. Since the term to calculate swale perimeter is somewhat complex, and the area term is much more important in terms of capacity and flow, the approach in this model is to simplify wetted perimeter as being equivalent to top width *w*.

As the exponent for *R* is less than 1.0, the impact of any difference is relatively small. At depths below a foot where biofiltration occurs, this slightly overstates *R*. Therefore, the $R^{0.67}$ term in Manning's equation will increase, overstating velocity and flow. Given that pollutant removal estimates are correlated with increased residence time (at reduced velocities), this introduces a conservative bias to the results. Since the area term is typically at well over one foot, and flow is a function of area squared, precision in the area term is much more important in terms of the overall accuracy of the computations.

Therefore, hydraulic radius *R* is assumed to be equal to A/*w*. Given (17), and wetted perimeter *P* = *w*, hydraulic radius is as follows:

$$R = dB/(B+1) \tag{20}$$

As in the case of cross-section area, it is thus possible to solve for hydraulic radius as a function of the one variable *d*.

6.4 SWALE ROUGHNESS

A key element involved in the design and function of biofiltration swales is the change in Manning's *n* as a function of *VR*, the product of velocity and hydraulic radius. Ree and Palmer (1949, as cited in NRCS, 1992) were the first to investigate this phenomenon empirically, and their work underlies the design basis of grass swales in the Engineering Field Manual (NRCS, 1992) as well as the design of biofiltration swales by Horner (1988). Ree and Palmer found that at a given *VR*, roughness increases as a function of vegetation density and height. Manning's *n* was highest at the lowest values of *VR*, decreasing as *VR* increased. Since Manning's *n* decreases as velocity increases, an increase in flow rate affects flow velocity more than flow depth. Laboratory studies (Kao and Barfield; 1978; Kouwen and Li; 1980, Wu and others, 1999) support the general relationships proposed by Ree and Palmer for emerged flow, when vegetation is overtopped and bent down by the flow.

However, when flow depths are submerged below vegetation height, these researchers noted substantial differences in the relationship between Manning's *n* and *VR*. Under these conditions, Manning's *n* increases in proportion to both slope and square of velocity toward a maximum at the transition to overtopping flow depths (Kuo and Barfield, 1978). Measurements of swale flow in Washington showed that the value of Manning's *n* increased 61 percent from 0.123 at 0.336 fps to 0.198 at 0.472 fps (SWPCD, 1992). When flow is thus submerged, an increase in flow rate affects flow depth more than flow velocity. This counterintuitive relationship was confirmed in measurements of swale flow in Washington, where an increase in flow rate of 55 percent increased velocity by 40 percent, while depth increased by 72 percent (SWPCD, 1992).

In the Horner (1988) method of swale design, Manning's *n* is manually derived by an iterative method comparing an estimated value of *n* and interpolated *VR* from the Ree and Palmer retardance curves, and recalculating *VR* with a different value of *n* if the selected value diverges by more than 5%. This requires several iterations, and it does not address values of Manning's *n* for submerged flow, which is not presented in the NRCS (1992) retardance curves of Ree and Palmer for the vegetation types used in biofiltration swales. As a result, a fixed value of 0.20 was recommended for Manning's *n* under submerged flow conditions (SWPCD). However, this approach results in a larger velocity response to differing flow rates, instead of the relatively small changes in velocity observed.

As an alternative to this method, it is desirable to generate a function relating Manning's *n* to *VR* for both flow regimes. Since a correct roughness value is critical in the design of biofiltration and conveyance swales (SWPCD, 1992), DURMM incorporates this function as part of the design worksheet. This relationship is expressed as a family of coefficients for retardance equations relating Manning's *n* as a power function of *VR*, using separate curves for submerged and emerged flow:

$$\text{for emerged flow, } n = C \times VR^{D}, \quad \text{for submerged flow, } n = E \times VR^{F} \qquad (21)$$

Both equations are solved simultaneously in DURMM, with the lower value of *n* used in subsequent computations. Table 6-1 below shows how these coefficients vary by surface cover:

Table 6-1: Surface Code/Retardance Coefficients

SURFACE TYPE		PAVEMENT	STONE	SHORT GRASS	DENSE GRASS	THICK BRUSH
SURFACE CODE	RETAR-DANCE	1 N/A	2 N/A	3 ("E")	4 ("D")	5 ("C")
C		0.0110	0.0240	0.0355	0.0600	0.0750
D		-0.080	-0.090	-0.360	-0.495	-0.570
E		N/A	N/A	0.800	1.150	1.900
F		N/A	N/A	0.500	0.540	0.600

Accuracy is improved by fitting this function to the range of *VR* below 1.0 involved in bioswale design. It is thus possible to closely approximate retardance curves (well within scaling accuracy) by choosing the appropriate values of *C*, *D*, *E* and *F*. Figure 6-2 presents a family of curves based upon these coefficients, such that each curve corresponds to a certain range of *VR* for a specific retardance.

Retardance values are scaled from the curves of Ree and Palmer as published in the Engineering Field Manual, and in Kao and Barfield (1978). Data points from flows in grass filter strips from observations by Abu-Zreig (2001) are included for comparison of these submerged flow values. It should be noted that the curves of Kao and Barfield (1978) do not show such a sharp transition from submerged to emerged flow, and slope also has a substantial effect on Manning's *n* that is not addressed in (21). (Curves for a 2% slope were used in Figure 6-2, since they corresponded most closely to the retardance curves at emerged flow, and bioswales should be designed at the lowest possible grade for best performance.)

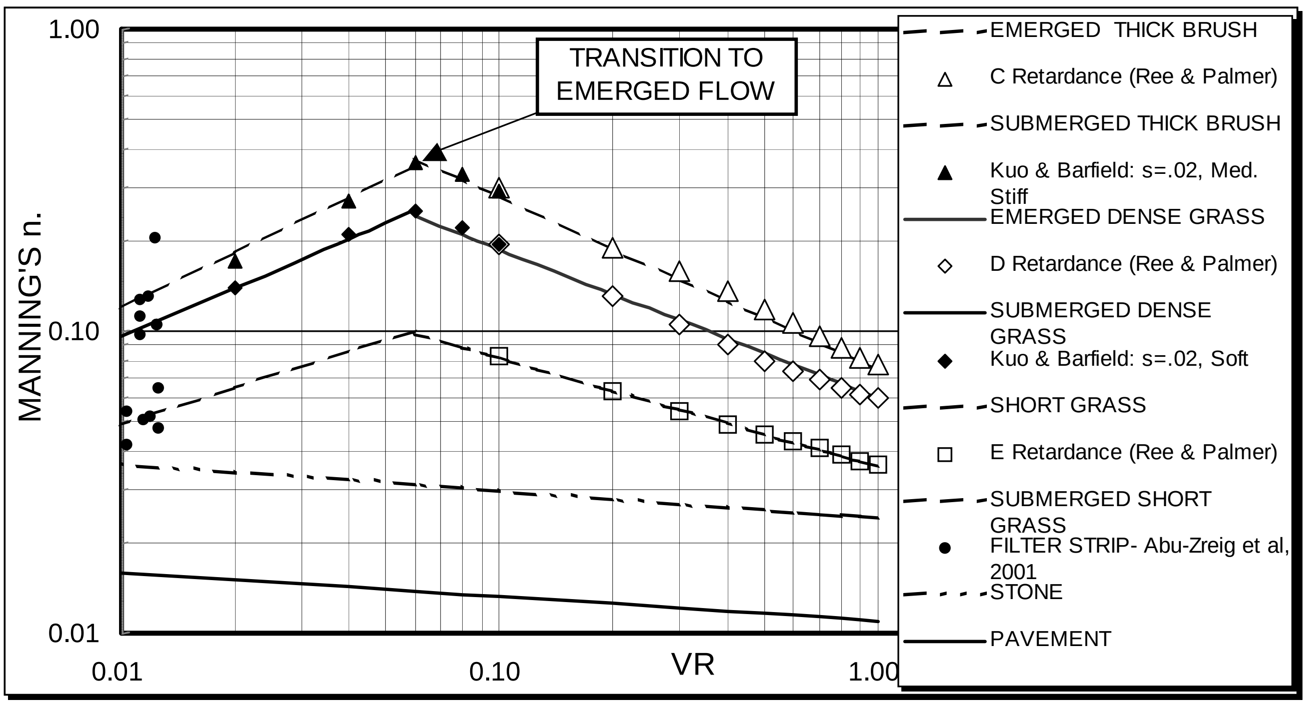

Figure 6-2: Relationship of Manning's *n* to *VR*
Sources: Ree & Palmer (1949), Kao & Barfield (1978) and Abu-Zreig (2001)

Nonetheless, this relationship provides flow velocities for submerged flow that fall within 10% of the flow velocity observations observed by the SWPCD (1992, Table 5-6). This relationship implies that, as flows increase under submerged conditions, flow velocities show minor increases while depth increases. Under emerged conditions, the opposite occurs, as flow depths show minor increases while velocities increase.

The dense grass curve corresponds to Surface Code 4 (or retardance "D") in the range of *VR* values between 0.02 and 1.0 (the Biofiltration Function in Horner's (1988) method). The short grass curve corresponds to Surface Code 3 (or Retardance "E") in values of *VR* between

0.02 and 1.0, representing retardance before grass is well established (the Stability Function). To account for higher retardance due to poor maintenance, the thick brush curve of Surface Code 5 is fitted to values of *VR* between 0.02 and 1.0 for retardance "C" (the Capacity Function). Note that the divergence from the retardance curves is well within 5% at *VR* values below 1.0 typical to biofiltration. Additional curves have been inserted for use on impervious surfaces, using roughness values of 0.11 and 0.24 at *VR* = 1.0 for pavement and stone, respectively. DURMM incorporates these differing surface types in determining flow conditions. Note that Manning's *n* can now be derived as a function of a single variable, depth *d*.

6.5 SWALE FLOW VELOCITY AND DISCHARGE

Given that area, hydraulic radius, and Manning's *n* are all expressed in terms of depth *d*, it is possible to solve for velocity in Manning's equation as a function of depth *d*. Manning's equation for open channel flow is expressed as follows:

$$V = \frac{1.486}{n} \times R^{0.67} \times s^{0.5} \tag{22}$$

where *s* is channel slope. Replacing the terms for *n* and *R* in (20) and (2i) respectively,

$$V = \frac{1.486}{C[VdB/(B+1)]^{D}} \times [dB/(B+1)]^{0.67} \times s^{0.5} \tag{23}$$

Rearranging terms,

$$V = V^{-D} \times \frac{1.486}{C} \times [(B/(B+1)]^{(0.67-D)} \times s^{0.5} \times d^{\,(0.67-D)} \tag{24}$$

Solving for *V*,

$$V = \left[\frac{1.486}{C} \times [(B/(B+1)]^{(0.67-D)} \times s^{0.5} \times d^{\,(0.67-D)}\right]^{(1/1+D)} \tag{25}$$

This equation is incorporated into DURMM for swale flow to determine flow velocity as a function of swale shape, surface type and depth of flow, *d*. Swale discharge is then the product of (19) and (25).

When swales are designed for shallow flow depths, the effective roughness values can exceed 0.200, resulting in flow velocities well below one foot per second. This can be particularly important when designing overland flow conveyance BMPs that depend upon residence time to function properly, and it minimizes resuspension of particles in larger events. It is also very important in increasing *Tc* when the dominant flow path is from impervious areas with a rapid sheet flow. Since these swale flow velocities are generally much lower than that allocated by TR-20 in shallow concentrated flow routines, swale flow becomes the controlling factor in determining *Tc*.

The assumptions inherent to equations (19), (20) and (21) introduce minor errors at flow depths above 0.5 feet. With an 0.8 foot depth in an 8 foot wide swale with 5:1 side slopes and a slope of 2%, the area term (19) is reduced by 1%, while hydraulic radius term (20) increases

3.2% and Manning's *n* term (21) decreases by 4.4%. This results in flows decreasing by 5.9%. However, using an example with a given flow of 28.7 cfs, depth decreases only 1.6% and velocity decreases 2.8%. Given that there is interpolation error and the method is quite precise below depths of 0.5 feet, these minor errors are considered acceptable for conveyance design for wide swales with shallow side slopes. However, it is not recommended for triangular channel sections at deep flow depths, where the errors in (21) can become unacceptably large.

Even when there is no defined channel, shallow concentrated flow is replicated as a swale with a wide bottom and very flat side slopes, as determined from the site plan. As *Tc* is recalculated for each runoff event, this results in decreasing *Tc* as *P* increases. By addressing the implications of swale design in such depth, DURMM provides for overland conveyance computations that are much more representative of actual conditions. Furthermore, these routines permit the designer to explicitly optimize swale design to retard flows and extend *Tc*.

Since pipe flow in the small areas involved in nonstructural BMP design is generally fairly short, and its average velocities are relatively high, the travel time involved is typically negligible. Therefore, DURMM usually omits pipe flow considerations in determining *Tc*. If necessary, long runs of pipe flow can be modeled as swale flow with steep side slopes and a paved surface. The velocities resulting from this approximation are quite close to that computed for pipe flow, so the travel time for a given length is quite similar.

The conveyance module of DURMM described in Appendix A provides estimates for swale response as a function of loading rate, length, width, side slopes, longitudinal slope, and surface cover. Consecutive segments of swale flow are provided to model more complex flow paths. Since the average swale flow is less than peak flow from the outlet, DURMM routines allow for direct input of the proportion of total flow that is conveyed by each swale segment. Design standards, construction specifications and details of biofiltration swales are set forth the Green Technology Standards, Specifications and Details, Appendix B.

6.6 SWALE STORAGE AND DISCHARGE THROUGH CHECK DAMS

Placed at regular intervals, check dams in bioswales can be used to create a series of cascading pools that will provide substantial detention. To estimate the storage volume required to meet predevelopment discharge rates, the following equation from TR-55 (Figure 6-1) provides a useful approximation:

$$V_s / V_r = 0.682 - 1.43_1(q_o / qi) + 1.64(q_o / qi)^2 - 0.804(q_o / qi)^3 , \qquad (26)$$

where V_s/V_r is the ratio of storage volume to runoff volume, and q_o / qi is the ratio of peak outflow to peak inflow. The coefficients are applicable to Type II and Type III precipitation. Given pre- and post-development flow rates and runoff volumes derived from the methods discussed above, DURMM computes the estimated storage needed in each event to meet predevelopment flow rates. Note that these are estimated storage volumes, and a comprehensive storage-indication routing method is necessary to confirm the storage volume required according to the outflow routing dynamics.

Since the storage depths in the pools are quite low, it is necessary to obtain stage-area and stage-discharge relationship information at quite small intervals. However, it is difficult to precisely obtain accurate stage/area relationships from construction plans without substantial interpolation errors. When the bioswale with check dam option is used, DURMM precisely calculates the stage-area and stage-discharge relationships for the swale/check dam system.

Given design parameters of longitudinal slope (*s*), bottom width (*w*) and side slopes (*ss*, hor./vert.), the expression for pool surface area at depth *h* where the pond created by a check dam does not extend up to the next upstream check dam is as follows:

$$A = \frac{h}{s} \times \left[w + (ss \times h) \right] \tag{27}$$

At higher ponding depths, check dam spacing is less than the calculated pool length, so the length term *h/s* is replaced by the length between the pools, or total bioswale length *L* divided by number of pools *n*. The average width term is also adjusted to account for the depth at the upstream dam location, as determined by slope *s* and spacing *L/n*, as below:

$$A = \frac{L}{n} \times \left[w + ss \times (2h - \frac{Ls}{n}) \right] \tag{28}$$

Using this relationship, DURMM computes the stage-area relationships at intervals of one half the check dam height for entry into the stage-area input fields of a separate hydraulic routing software package. By entering various bioswale lengths, slopes and number of dams, this procedure makes it possible to calculate the increase in volume provided by closer spacing and/or higher check dams.

Equations (27) and (28) are also applicable for calculating storage within terraces with differing side slopes, using the average of the two differing side slopes and a bottom width *w* of 0 feet. Where terraces are designed to provide biofiltration, they should have a bottom width required for biofiltration. This will normally require excavation into the native soils, resulting in the more uniform side slopes typical to bioswale.

Given stage-area relationships, it is then necessary to route flows through and over the check dams. Complex (and expensive) prefabricated structures are not the optimal arrangement to control flows through check dams, particularly where a very small orifice needed for quality control would be prone to clogging. As a cost-effective alternative, stone-filled gabions can be designed to provide both the required check dam geometry, as well as a mechanism for conveying low flows through the dams. Smaller flow events are routed through the stone, while an orifice and/or weir in the center of the dam is designed to route the larger events. Actual check dam routing is accomplished by entering the dynamics of the check dam design in a separate hydraulic routing software package.

DURMM provides a method to calculate flows through the stone used to construct check dams. Flow through the stone is calculated as a function of stone size, flow depth, width of flow and flow path length. These relationships were initially investigated by McIntyre (1990), who examined gabion weirs as a measure to control flows released from detention basins. McIntyre found that unit width flows could be calculated between these variables according to the following relationship:

$$q = A \times \left(h^B \middle/ L^C \right), \tag{29}$$

where q is the flow per unit width, h is the ponding depth, and *L* is the length of flow (up to two feet). A, B and C are coefficients based upon stone size (small, medium and large). Over flow lengths of 1 to 2 feet, (29) provided a very good relation (r^2 = 0.992 to 0.998) for the media evaluated, given the proper values for coefficients A, B and C.

However, the variation in these coefficients does not follow a straightforward relationship to stone size, so (29) cannot be applied for stone media falling outside of the range of stone sizes evaluated. Also, measurements of the flow profile as it passed through the stone by McIntire et al (1991) indicated that entrance losses were significant, so a different form of the denominator of equation (29) was required. Equation (29) was thus modified to address entrance losses, longer flow lengths and different size stone sizes according to the following relationship (McIntyre et al, 1991):

$$q = \frac{h^{1.5}}{(L/D + 2.5 + L^2)^{0.5}}, \tag{30}$$

where *D* is the average stone diameter in feet. The stone gradation is intended to be relatively uniform in size. Designed for flow paths of up to 6 feet, (30) addresses the range of flow lengths involved in check dam flows. Note that, where the length *L* is short (less than several feet), changes in stone diameter have a relatively greater effect than where flow paths are longer. Flow path length becomes the dominant factor once it exceeds several feet

The typical gabion width is three feet, so this would be the maximum flow length in most designs. However, where the discharge requirements require a stone size in the range of 1 to 2 inches, such stone will not remain inside a gabion with a mesh of 3 inches. To resolve this, the exposed portion of the gabion can be filled with 4-inch rock, while the smaller stone is placed inside. This reduces the effective flow length to 2.3 feet or so, depending upon the average diameter of the larger rock. Even with the smaller flow length, the smaller stone will result in lower discharge rates. If river rock is used for the exterior stone, this type of gabion dam can be quite attractive.

At low flows, the average flow width through the check dam is less than that at high flows, where weir flow dominates the total flow response in most designs. The geometry of the swale width and side slopes thus becomes the dominant determinants of flow routing at a given stone size. The effective flow width *W* is calculated as being the bottom width *w* of the swale, to which is added ponding depth *h* times swale side slopes *ss*:

$$W = (ss \times h) + w, \tag{31}$$

DURMM also permits manual entry of a flow path width if a design section narrower than the full width of the swale is needed to control quality events. Since this can be more difficult to construct, this approach is not recommended unless strict release rate criteria are required.

Given the stage-area and stage discharge relationships, it is thus possible to route flows through the bioswale/check dam system. Usually, the combined volume from all ponds can be routed as if they were one pond, discharging through the outflow design for the lowest pond. In check dams where the final outlet is a structure such as a catch basin, it is best to model that pool as a separate pond, since its outflow dynamics can differ substantially from that found in the check dams.

Tailwater will play an important part in routing flows through the check dams, as tailwater retained in the downstream pools will attenuate the flows through the check dams, particularly during conveyance and flooding events. Assuming that the pools tend to fill and discharge together (see below for more discussion), once the flow depth in the downstream pool has backed up onto the stone, increases in flow depth will not increase the head through the stone. For this reason, DURMM provides stage-discharge data through the stone only up to the pool depth, since it will be essentially constant from then on. This data is entered as a special outlet into a separate hydraulic routing package.

For controlling the larger flows, routing software should be used to determine the dynamics of other weirs and orifices. A simple control structure is to install one or several PVC pipes above the quality storm event elevation, sloped so they discharge at grade on the downstream side. Modeled as culverts with entrance losses, this arrangement is not only inexpensive, it also reduces potential scour at the pipe discharge.

During a runoff event, the upper pool fills up first, increasing its effective discharge relative to the lower pools. Since the peak discharge volume will be more than the individual pool can attenuate by itself, it often will overtop the entire dam by several inches. Therefore, the check dam should be depressed by at least 6 inches in the center to ensure flows do not erode the side slopes of the swale. This peak elevation often occurs after peak discharge, since the lower pool is subsequently filled up, thus creating more tailwater and reducing structure efficiency. As the discharge peak passes through the pools, the upper pools also drain more rapidly than the lower pools, reducing their effective head. This relationship tends to distribute the storage volume more rapidly at the beginning, and delay its release at the end of the event.

Tailwater thus varies throughout the event, and until recently, modeling it accurately in a series pool configuration is beyond the capability of most hydraulic routing software packages available. Where precise routing is required, each pool should be modeled individually using software capable of dynamic routing. If routed as individual ponds, these trends result in slightly poorer peak rate reductions of the swale/check dam system when compared to being routed as one larger pond. However, the peak discharge is delayed for longer period, which can be beneficial by desynchronizing peaks from other subareas.

CHAPTER 7 BMP POLLUTANT REMOVAL PROCESSES

7.1 POLLUTANT REDUCTION PROCESSES

Overland flow BMPs remove pollutants from urban runoff through six major pathways: infiltration, filtration, adsorption, immobilization, settlement and transformation. Some, or even all, of these processes can occur simultaneously, depending upon the pollutant involved and the type of BMP. However, there are very few BMP design tools that explicitly account for how these processes function in overland flow BMPs, let alone project the performance to be expected from a particular design.

In wet retention ponds, settling equations have been applied to TSS and particulate nutrient removal (Ferrara and Hildick-Smith, 1982). In wetlands, first order equations are used to project removal of nitrogen (Kadlec and Knight, 1996), and more complex equations have been proposed for removal of phosphorus (Dortch and Gerald, 1995). The complex processes involved in metals removal by wetlands have been investigated in detail by Kallin (1999). Intricate models that simulate these processes have been formulated by several researchers (Kallin, 1999; and sources cited therein).

In the case of vegetated filter strip BMPs, the VFS-MOD model by Munoz-Carpena and others (1992) is an effective tool to predict suspended sediment removal efficiency. A windows-based program, VFS-MOD is an excellent tool to estimate filter strip performance for TSS removal. Recently, Rudra and others (2002a 2002b) have added a phosphorus component, but it is not available yet. While VFS-MOD is quite easy to use, it requires more data entry than typically applied at the typical site development level, and it does not address other pollutants.

The common thread to all process-based models is that the dynamics of pollutant removal reflect the interactions between pollutant loads, hydrologic factors and BMP design parameters. However, such process-based models are too complex for widespread use at the site level. As a result, the approach generally taken by the regulatory community is to assume that a particular BMP imparts specific pollutant reduction efficiencies, assuming certain minimum dimensional and/or loading criteria are provided.

Unlike retention basins however, there is very little data on filtering BMPs (USEPA, 1999). By assuming that the performance from such a limited dataset should be globally allocated to a particular BMP for every case, there is a high likelihood that the "design" pollutant reduction efficiency to be applied will be incorrect. This is analogous to using a broken clock, which can still tell time accurately twice a day. The designer can only hope that the "time" setting is close to what is realistic. Furthermore, this approach provides neither method nor incentive to optimize BMP design for the particular circumstances of each site. Thus, besides being probably inaccurate, this approach thus neither rewards a good design, nor penalizes a bad design.

On the other hand, it is readily apparent that, the better the design is, the better the removal rate will be, and vice versa. To advance the process of BMP design for the regulatory community, the intent in DURMM is to provide a simplified approach that generates results close to that anticipated from process-based models, while requiring much less intensive input data. While DURMM does not attempt to replicate the actual processes involved, it does take into account how the design parameters of overland filtering BMPs respond to their influent pollutant and hydraulic loads.

This approach is far preferable to assuming an invariant pollutant removal efficiency for each BMP. To carry out this approach, the literature on BMP performance has been examined to derive trends that can be represented by relatively simple equations based upon design parameters. Essentially, the concept is to “observe and regress”. In this manner, the design of overland filtering BMPs can be better tailored to address the particular situation. Although DURMM is an event-based model, if the relationships derived from this process reflect annual loads and annual flow-weighted concentrations, the results are more likely to better reflect BMP performance through the year.

Unfortunately, there is very little data that actually provides the basis for such relationships. Even though there have been many studies of pollutant removal efficiencies, these studies sample different pollutants, use different sampling methods, have very different input loadings, have many different design parameters, and they present their results as the “average” of the differing pollutant reductions observed in many events. Rare are the studies that investigate annual loads to develop flow-weighted average concentrations, as discussed in Chapter 3.

Depending upon the pollutant measured and the BMP, reported reduction percentages can range over an order of magnitude, and negative reduction percentages are often reported. The issue is further confounded by the predilection for total nitrogen and total phosphorus to be reported as if they were single pollutants, when in fact they each comprise several compounds with very different characteristics affecting their removal by BMPs. This latter aspect is particularly important in overland filtering BMPs. As a result of these confounding factors, published summaries of BMP performance report a considerable variation in the efficiency of BMPs to remove pollutants (CWP, 1997; ASCE, 1999; USEPA, 1999).

The following analysis of BMP performance examines the percentage reduction in effluent EMCs by BMPs, known as the efficiency ratio. Efficiency ratio is <u>not</u> equivalent to the removal percentage expressed as a reduction in pollutant mass loads, as is reported in most studies. This is due to the fact that mass loads represent the product of effluent EMCs and effluent volumes. Since effluent volume losses are considerable, and they are accounted for in the hydrology/hydraulics analysis, it is the EMCs of pollutants remaining in the surface runoff conveyed from the BMPs that become the parameter of interest. Multiplied by the computed runoff volumes leaving the BMP, DURMM then develops the reductions in mass loads during the quality runoff event.

The resultant approach thus follows the sum of loads method presented by Strecker and Quigley (1999). This method is more appropriate for the overland BMPs discussed in this Manual, since infiltration losses can be significant, and efficiency ratios by themselves would understate BMP performance. Given dilution by the receiving stream in any event, total loads are also more accurate in estimating runoff contributions to instream pollutant loads as part of

TMDL requirements. It follows that if reduction targets are met in the 2.0 inch quality event, they will be met in all runoff events that are smaller in volume.

To resolve the factors involved in the reported inconsistency in BMP efficiencies, DURMM examines three factors that control much of the variability in BMP performance. These factors are: 1) input EMC, 2) minimum irreducible EMC, and 3) potential maximum efficiency ratio. Note that input concentrations are independent of BMP design, and minimum irreducible EMC and potential maximum efficiency ratio are generic to the type of BMP. Thus the final value for efficiency ratio is related to effectiveness in the design of the BMP itself. To refine estimates of efficiency ratios, total nitrogen and phosphorus are also segregated into the various species that affect their removal by BMPs.

7.2 IRREDUCIBLE CONCENTRATIONS

It has been recognized for some time that BMPs and wastewater treatment wetlands cannot reduce pollutants below certain thresholds. When an input EMC is very low, such facilities may actually release sequestered nutrients and sediments, resulting in negative efficiency ratios (Strecker and Quigley, 1999). Claytor and Schueler (1996) examined this concept of irreducible (or background) concentrations, and reviewed the existing wastewater wetland and urban runoff BMP literature to obtain the minimum concentrations listed below in Table 7-1:

Table 7-1: Irreducible Concentrations (mg/l) in Urban BMPs

PARAMETER	TSS	TN	TKN	NO3	TP
Urban Runoff BMPs (Schueler, 1996)	20-40	1.9	1.2	0.7	0.15-0.20
Wastewater Wetlands (Kadlec and Knight, 1996)	2-15	1.0-2.5	1.0-2.5	0.05	0.02-0.07

These concentrations will be evaluated in depth for each BMP in the following sections. Irreducible concentrations are obtained from the literature by searching for the lowest repeated outlet EMC reported for a given type of BMP. Given that DURMM provides input values of the EMCs as discussed in Chapter 3, two of the three variables that control performance are thus readily available before even beginning design of a BMP. The remaining element is to determine the potential maximum efficiency.

7.3 MAXIMUM EFFICIENCY RATIOS

Bell and others (1995) measured efficiency ratios as a function of input concentration. As displayed in Figure 7-1, total phosphorus efficiency ratios from a sand filter in Alexandria, VA, showed an increasing trend with input concentration, with the highest efficiency ratio at 87%. It is possible to replicate this relationship by using an asymptotic function in the following form, where efficiency ratio *R%* is related to the design maximum efficiency ratio R_{max}*%*, and the input EMC multiple *M* of the irreducible concentration:

$$R\% = R_{\max}\%(1 - e^{-K(M-1)}) \quad (32)$$

The lowest effluent concentration reported was 0.065 mg/l, given 35 percent removal at 0.01 mg/l. This suggests that irreducible concentration would be in the range of 0.050 mg/l, shown as the minimum on the upper axis. Using this value for the irreducible concentration, a value of 0.25 for K, and a design maximum efficiency ratio of 90%, the efficiency ratio curve at the higher multiples is within 4 percent of the values reported by Bell and others (1995). The upper x axis displays the input EMC, while the lower x axis displays the multiple *M*. The ordinate displays efficiency ratios, with an asymptote at the maximum of 90%.

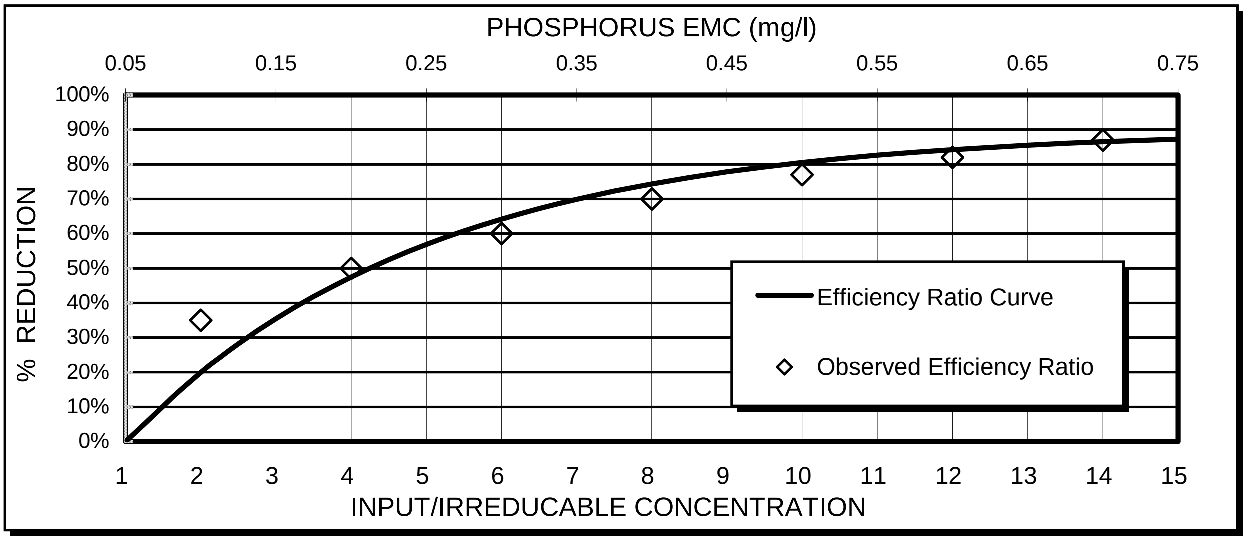

Figure 7-1: Sand Filter Efficiency Ratios as A Function of a Maximum Efficiency Ratios and Input EMC Multiple of the Irreducible Concentration. Source: Bell and others (1995)

Note that this estimated value of the irreducible concentration for TP falls well below the range reported by Claytor and Schueler (1996) for runoff BMPs. In fact, this value is at the low end for wastewater treatment literature. Since (32) underestimates efficiency ratios at multiples below 4, this trend suggests that the minimum may be even lower (which would give higher multiples at these input EMCs). Even though (32) understates the efficiency ratio at low multiples, it is the higher range of multiples that is most relevant to BMP design, so this underestimation has minor effects on the utility of DURMM approach.

Equation (32) thus presents the relationship for projecting likely efficiency ratios as both a function of the irreducible concentration and input EMC. This relationship underlies all further projections of the efficiency ratios as a function of $R_{max}\%$. Note that *(32)* is identical to the standard diffusion equation used in modeling wetland nitrogen removal, when the K value is adjusted to reflect the concentration involved:

$$R\% = (1 - e^{-K(EMC - C\min)}) \qquad (33)$$

Unlike the study displayed in Figure 7-1, most of the literature on BMP performance omits the individual event data needed to plot efficiency ratios as a function of input EMC. Therefore, it is difficult to derive the appropriate value for the coefficient K for other BMPs. In the best possible case, a multiple of at least 9 would be required to obtain a maximum removal efficiency of 90%. Using the K value of 0.25 shown in Figure 7-1, it is seen that maximum removal percentage occurs at a multiplier of at least 14. This suggests that appropriate K values can be related to maximum removal efficiencies by assuming that the maximum efficiency ratio occurs at an EMC of roughly 15 times the irreducible concentration. In this case, the value of 0.25, as used in Figure 7-1, is applied as the default value for all BMPs. This value should be refined as more data for individual BMPs becomes available.

Too often, total N or total P efficiency ratios are the only criteria reported, with maximum rates typically below 50%. However, in many overland BMPs, efficiency ratios for soluble fractions will be minimal, while efficiency ratios for the particulate fraction are typically much higher. In contrast to the considerable cycling between these forms that occurs in wet basins and wetlands where the retention time is quite long, there is less cycling between these forms in overland filtering BMPs with relatively short retention times. This is the reason that nutrient loading in Chapter 3 is partitioned into nitrate, ammonia, organic nitrogen, soluble phosphorus, and particulate phosphorus fractions. By segregating nutrients into these fractions, it is possible to differentiate potential reduction of each fraction according to the simplified processes involved in each BMP. Each nutrient fraction will have different irreducible EMCs, with resulting efficiency ratios that vary from minimal to high.

For instance, vegetative filters are relatively ineffective in reducing concentrations of soluble phosphorus, while particulate phosphorus can be effectively filtered. If the relative contributions of the soluble and particulate fractions are available from the loading EMCs, it is possible to obtain better results of the total nutrient reductions. The literature for each BMP is analyzed in the following sections to determine the values for minimum concentrations and maximum efficiency ratios. Note that these values should reflect results supported by other values in the same range. Extreme values that are isolated are not a reliable basis to determine these values.

Maximum efficiency ratio is thus dependent on the pollutant involved, the type of BMP, and how well it is designed to address its loading rates. The reduction processes and the particulars involved in optimizing BMP design are addressed for each type of BMP in the following sections. Relevant relationships of factors such as hydraulic residence time, hydraulic loading ratios, surface/volume ratios, length/width ratios, depth of flow and other parameters are examined for each BMP to develop a quantitative approach to project efficiency ratios for each fraction as set forth in Chapters 9, 10, and 11.

CHAPTER 8 INFILTRATION BMPs

8.1 INFILTRATION BMP POLLUTANT REMOVAL MECHANISMS

The issue of potential leaching of toxic pollutants and nutrients from infiltration BMPs into groundwater has been raised in Chapter 3. While there may be less opportunity for transformation and microbial immobilization processes than in a vegetated surface BMP, filtration processes will reduce pathogens and particulate pollutants, and adsorption processes can sequester soluble pollutants. In essence, the subsoil under the infiltration BMP can function as a sand filter in retaining pollutants in the soil profile. However, since the subsoil profile varies greatly in terms of porosity and cation exchange capacity (CEC, a measure of adsorption potential), the potential for infiltration BMPs to avoid contamination of groundwater is also highly variable.

In sandy soils with a low CEC, the potential for adsorption can be quite low, while infiltration rates would be high. On the other hand, finer soils with a high content of clays will provide excellent filtration and adsorption, but infiltration rates will be lower. Since infiltration BMPs are generally designed for higher rates, it follows that the potential for filtration and adsorption will be relatively low in most infiltration BMPs. For this reason, infiltration BMPs are not recommended for urban "Hot Spots", unless runoff is already well filtered by other BMPs (Parmer and others, 1995). In general, if they are located in mineral soil a sufficient distance from groundwater (at least 20 feet) there appears to be minimal concern for groundwater contamination, even with highly polluted runoff (Livingston, pers. comm.). However, in sandy soils and/or where depth to groundwater is shallow, runoff must be pretreated to ensure that dissolved toxins do not migrate directly into the groundwater.

In terms of nitrogen species, ammonia in urban runoff is either adsorbed to the soil particles, or immediately nitrified into nitrate in the oxic zone under the BMP (Gold and Sims, 2000). Much of the adsorbed ammonia will eventually be nitrified. The extent that biogeochemical processes then sequester nitrate in subsoil is much less than found at the surface, where biomass accumulation can be substantial in the root zone. This results in the soluble species of nitrogen leaving the BMP as nitrate. Since nitrate passes through infiltration BMPs into groundwater without transformation, adsorption or uptake, loading of nitrates generally increases in proportion to the reduction in loading of ammonia.

Where ammonia in the receiving waters is more of a problem than nitrate enrichment, as is occasionally the case for upland streams, this is an effective approach to minimize ammonia loading. However, since nitrates will eventually reach the estuaries and inland bays, this approach can have adverse consequences in the long run. Since urban runoff pollutant loads are segregated into both of the soluble forms, as well as the particulate, DURMM could be used to project event subsurface loading of nitrate and ammonia by assuming that most of the ammonia load is transformed into nitrate. Organic N does not seem to leach into groundwater, so leaching losses of organic N should be minimal. Since these processes are either all or none, there are no process-based approaches to determine the extent of load reductions into groundwater.

Even under very high loads from septic systems, nearly all of the soluble orthophosphate is adsorbed within the soil profile (Gold and Sims, 2000), unless the soils are very sandy. Phosphorus losses would be even less for the particulate forms. However, as they accrue over the long term, some of the particulate fractions will be transformed into soluble forms. Leaching of phosphorus in the groundwater has recently been recognized as a contributor to nutrient loads (Sims and others 1998). However, since these issues are so site specific, subsurface losses cannot be reliably estimated by an event-based approach, so they are not addressed in DURMM.

Therefore, even though infiltration BMPs can substantially reduce surface loading of nutrients and other pollutants through volume reduction, this does not necessarily represent a corresponding reduction in the total nutrient load to streams. This is particularly applicable to nitrate, since there are generally minimal transformations once nitrate passes below the root zone. Even though dissolved nutrients may be out of sight, they may not be out of the regional groundwater system, and thus they represent potential loading from site development.

As a result, groundwater loading from infiltration BMPs could result in significantly less reduction of the amount of soluble nutrients than that reported in the literature, where subsurface losses are rarely addressed. In the absence of a rigorous basis to estimate nutrient and pollutant transformations and adsorption processes in the subsoil, the current approach in DURMM is to ignore these losses.

8.2 SUBSURFACE INFILTRATION BMPs

Infiltration trenches are the preferred subsurface nonstructural infiltration BMP. Dry wells connected to roof downspouts are another common infiltration BMP, but they suffer from an absence of pretreatment. Infiltration basins are structural BMPs that require considerable area and maintenance, and they often fail after several years of operation. DURMM thus focuses on infiltration trenches as the main nonstructural BMPs for site design.

For proper operation, infiltration trenches must be designed to ensure that they do not clog over time. This requires that filtering BMPs are used to provide adequate pretreatment, designed according to the principles set forth in the following chapters. Where the dissolved forms of nutrients, metals and PAHs are a concern due to elevated levels in runoff, pretreatment by these BMPs is essential in any event. As shown in Chapter 3, runoff from roofs can have substantial amounts of TSS, zinc and nutrients; if connected directly to a dry well, the facility is likely to clog, and groundwater loading of nutrients and zinc will occur. While an in-line catch basin can reduce TSS loads, it has minimal effect on nutrients, and it is likely to be poorly maintained, leading to eventual failure. For these reasons, direct connection of downspouts to dry wells is not recommended unless measures can be established that ensure adequate maintenance.

Infiltration trenches thus are the preferred nonstructural subsurface infiltration BMP. Note that volume to surface area relationships and the interaction between width and depth to SHWT favor trenches that are long and narrow over shorter, wide trenches (Guo, 1998). This is another reason to avoid dry wells and other rectangular/circular subsurface infiltration structures. However, note that the preferred geometry can be easily incorporated into linear BMPs such as biofiltration swales and bioretention facilities, which also serve to provide the required pretreatment. This is discussed in more detail in Chapters 10 and 11.

As to the appropriate trench material, crushed stone segregated into one size provides the highest void ratio and least potential for clogging, although a finer material can be used. Coarse filter fabric to prevent the entry of soil fines into the trench should surround the rock at the sides. Recent studies suggest that drainage fabric is not necessary at the top if a smaller stone is used, covered with a sandy loam (Covington, pers. comm.). It is important that the effective pore space of the fabric be quite large, as specified for drainage fabric. Coarse sand should be placed at the bottom to prevent subsidence of the rock into the soil. While clean washed stone is always specified, the reality is that it is rarely provided, so there is a real potential for rock dust to clog the interstitial pores in the adjacent soil matrix and filter fabric. Design standards, construction specifications and details for infiltration trenches are provided in Green Technology Standards, Specifications and Details, Appendix B.

In the case of subsurface infiltration BMPs, there is no reduction in EMCs in runoff that is not infiltrated. Surface runoff loads are only reduced in proportion to the amount of runoff infiltrated during the quality storm event. The inflowing EMC is multiplied by remaining surface runoff volume to determine mass loads leaving the site in surface runoff. The BMP design module of DURMM described in Appendix A provides estimates for infiltration structure performance as a function of loading rate, pollutant levels, length, width, depth, and infiltration rate.

8.3 SURFACE INFILTRATION BMPs

Infiltration has a substantial effect on surface runoff and its pollutant loads by reducing runoff volumes from filter strip, bioswale and bioretention BMPs. As runoff flows from impervious surfaces over the pervious surfaces of these BMPs, its volume can be substantially reduced during the quality storm event. This mechanism is implicitly incorporated into the mass load reduction calculations for filter strip, bioswale and bioretention BMPs as a function of the reductions in runoff volumes due to disconnection. Infiltration also is incorporated into internal site design in terms of impervious area disconnection, as discussed in Chapter 5.

Subsurface leaching losses of nutrients and toxic pollutants are negligible with impervious area disconnection and filtration BMPs, since an intact root zone and soil profile is present to intercept and sequester nutrient and pollutant loads. (See Chapter 3.) Therefore, these infiltration practices are not only very effective; they also have the least potential for groundwater nutrient losses and contamination. Furthermore, they require minimal cost to construct, and the required footprint of surface infiltration BMPs can be incorporated into the overall site layout. Since open areas suitable for surface filtering BMPs are typically provided between buildings and within parking lots in any case, these BMPs do not require excessive loss of potentially useable ground.

For this reason, DURMM goes to some length to determine the extent of runoff reduction in these BMPs, and that resulting from source area disconnection. The input data fields segregate impervious source areas into connected and disconnected categories, as well as receiving pervious areas. Runoff volumes are obtained from both types of impervious areas, to which is applied the impervious disconnection subroutine onto the receiving pervious areas, be they formal BMPs for connected areas or internal landscaping for disconnected areas.

After reducing EMCs in impervious runoff according to equations developed for overland filtering BMPs in the following sections, and multiplying the resulting EMCs by the reduced runoff volumes, mass loads can be substantially reduced. This approach is used for both disconnected internal flow paths and impervious surface runoff directly connected to BMP. The procedures involved in routing these flow paths are described more fully in the DURMM Users Manual, Appendix A.

CHAPTER 9 FILTER STRIP BMPs

9.1 SHEET FLOW FILTRATION RESULTS

Filter Strips represent a class of BMPs in which runoff passes as sheet flow through vegetation. Filtration BMPs provide reductions of pollutant loads through filtration by vegetation and infiltration. While it would seem that the best filter media is turfgrass, recent reports suggest that a dense native meadow stand can be similarly effective. Turf is the obvious choice for many filter strips, since they can be incorporated as lawn areas that happen to be specifically designed to intercept runoff from buildings and parking areas.

However, native warm season grasses have much deeper rooting systems than turf-type cool-season grasses. This greatly promotes infiltration and recharge of runoff into groundwater. There are also several native grasses that form a dense stand at maturity. Studies of a native grasses filter strip by Schultz and others (1993, as referenced by Prairiesource, 1999) show that native grass filter strips have much higher infiltration rates and vegetative uptake than turf grass. Meyer and others, (1995) demonstrated how a hedge of switchgrass as short as only <u>one foot</u> was able to trap nearly 80% of the total sediment load. Sediments settled out in the water ponded over a foot deep behind the grasses, and trap efficiency was higher for the coarse fractions. Since infiltration and runoff volume reduction is a fundamental BMP (see Chapter 5), native warm season grasses are thus preferable for filter strips. However, it is important that the plantings be as dense as possible for the initial 15 feet, where filtering is most important.

Since filter strips are long and narrow, note that the discussion in this Chapter defines width as the dimension direction parallel to the flow, and length as the direction perpendicular to flow. It would seem that flow velocities through filter strips would be slower than flow through swales; however, the discussion on swale roughness in Chapters 6 and 10 suggests that flow velocities are similar in sheet flow and in swales at very low flow depths. Assuming that retention time is thus not the controlling factor, this suggests that grass filter strips provide a greater reduction in particulate fractions than swales since flow is restricted to the densest thatch and blades.

Unfortunately, most of the filter strip literature examines agricultural runoff conditions, where sediment loading rates are typically at least an order of magnitude greater than found in urban runoff. Using rainfall simulators at very high precipitation rates of 200 mm over a day, Dillaha and others (1989) reported TSS EMC reductions from 49% to 93%. Table 9-1 displays results from this site in the Virginia Piedmont for filter strips 4.5 and 9.1 meters wide over slopes of 11%, 16%, and 5% with a cross slope of 4%.

TABLE 9-1: Virginia Piedmont Filter Strip Load and EMC Reductions
Source: Dillaha and others, 1989

SOURCE AREA	WIDTH (m.)		18.3		LENGTH (m.)		5.5	
SLOPE (%)	11		16		5+4		ALL METHODS	
PRECIP. (mm)	200		200		200		200	
LOAD (cu.ft./ft.))	13.95		10.99		11.78		8.25	
RUNOFF (mm.)	70.8		55.8		59.8		186	
RUNOFF (l.)	7126		5616		6019		18761	
INPUT LOADS (kg/ha)								
TSS	3920		8940		2110		14970	
NH4	1.53		4.28		0.76		6.57	
NO3	1.65		1.98		1.22		4.85	
ON	11.73		26.57		6.92		45.22	
TP	4.34		8.41		2.27		15.02	
SP	0.19		0.17		0.10		0.46	
PP	4.15		8.24		2.17		14.56	
OUTPUT LOADS (kg/ha)								
WIDTH (m.)	4.55	9.1	4.55	9.1	4.55	9.2	4.55	9.2
RUNOFF (mm.)	66.1	17.9	55.8	53.2	16	16.6	137.9	87.7
RUNOFF (l.)	8307	2698	7013	8017	2011	2502	17331	13216
TSS	560	100	4,220	2,720	360	150	5,140	2,970
NH4	0.65	0.17	3.56	2.48	0.19	0.11	4.40	2.76
NO3	1.62	0.36	1.85	1.55	0.34	0.34	3.81	2.25
ON	2.91	0.78	12.76	10.11	1.04	1.34	16.71	12.23
TP	1.18	0.32	4.32	2.95	0.35	0.31	5.85	3.58
SP	0.27	0.08	0.17	0.25	0.04	0.04	0.48	0.37
PP	0.91	0.23	4.15	2.70	0.31	0.27	5.37	3.20
INPUT CONCENTRATION (mg/l)								
TSS	5537		16022		3528		8031	
NH4	2.16		7.67		1.27		3.52	
NO3	2.33		3.55		2.04		2.60	
ON	16.57		47.62		11.57		24.26	
TP	6.13		15.07		3.80		8.06	
SP	0.27		0.30		0.17		0.25	
PP	5.86		14.77		3.63		7.81	
OUTPUT CONCENTRATION (mg/l)								
TSS	679	373	6057	3415	1802	604	2985	2262
NH4	0.79	0.63	5.11	3.11	0.95	0.44	2.56	2.10
NO3	1.96	1.34	2.66	1.95	1.70	1.37	2.21	1.71
ON	3.53	2.91	18.31	12.69	5.21	5.39	9.70	9.31
TP	1.43	1.19	6.20	3.70	1.75	1.25	3.40	2.73
SP	0.33	0.30	0.24	0.31	0.20	0.16	0.28	0.28
PP	1.10	0.86	5.96	3.39	1.55	1.09	3.12	2.44
EMC REDUCTION								
TSS %	88%	*93%*	62%	79%	49%	83%	63%	72%
NH4 %	64%	*71%*	33%	59%	25%	65%	28%	40%
NO3 %	16%	*42%*	25%	45%	17%	33%	15%	34%
ON%	79%	*82%*	62%	73%	55%	53%	60%	62%
SP %	-22%	-11%	*20%*	-3%	-20%	4%	-13%	-14%
PP %	81%	*85%*	60%	77%	57%	70%	60%	69%

The extent of reduction was dependent upon filter strip width and slope; the longer and flatter, the better. Runoff decreased substantially in the flatter filter strips as it passed through them, so total load reductions were even greater. Note that the effluent TSS EMCs were still in the range of higher values reported for urban runoff EMCs, since input load EMCs were so high.

Hydraulic loading rate is defined as the runoff volume divided by the length of the filter strip in terms of cubic feet of runoff divided by linear feet of filter strip length normal to the flow. Hydraulic loading rates ranged from 11 to 14 cubic feet of runoff distributed over each foot of filter strip length. These values are at the upper end for typical BMPs, where the loading rate from a 1.5 inch storm over a 60 foot wide contributing impervious area amounts to 7.5 cubic feet per foot. As such, the reported rates represent up to 3 inches of runoff, which is greater than that required for quality storm design.

Similar reductions were noted in the organic N EMCs, while ammonia reductions ranged from 25% to 71%, and nitrate reductions were 16% to 45%. It was thought that nitrate reductions reflect dilution of runoff by rainfall on the filter strip. The authors noted that N transformations in vegetated filters can release sequestered organic N as nitrate, ammonia and, to a lesser extent, dissolved organic N, which was 5% of the total organic N. Ammonia reductions were greater than nitrate, since ammonia is adsorbed onto clay particles, while nitrate is very soluble. Soluble P reductions were minimal or negative, while particulate P reductions ranged from 57% to 85%.

These results support the partitioning of urban runoff into particulate and soluble fractions. Particulate fractions consistently showed high reductions in EMCs, while the soluble fractions -excluding adsorbed ammonia- showed minimal losses, or even increases. Note that good results were obtained with the shorter filter strip of 15 feet. This was confirmed by field observations showing that most of the sediment deposition occurred within the first meter or so. Even though the remaining length of the filter strip had much less accumulation, better results for the 30 foot filter suggest that runoff is further polished as it passes through the strip.

Using a similar experimental design with rainfall events, Parsons and others (1993) conducted a series of observations in the North Carolina Piedmont with filter strips of 5.2% and 6.3% slopes. Table 9-2 displays a summary of some of the storms monitored in this study. Excluding the storm of day 228 (a two year event), hydraulic loading rates are similar to an urban BMP.

As in the case of the results of Dillaha and others (1989), input EMCs of TSS were very high, as were the output EMCs. Although the loading rates were often in the range of urban runoff BMPs, TSS reductions were less. If data from the heavy storm of day 228 are removed from the totals, average reductions of TSS are in the ranges reported by Dillaha and others (1989). However, losses of ammonia and soluble P were much greater, with negative efficiency ratios for these nutrients. (Cells are left blank where a constituent EMC was greater than its combined species, eg., where TKN was less than ammonia.)

TABLE 9-2: North Carolina Piedmont Filter Strip Load and EMC Reductions.
Source: Parsons and others, 1993

SOURCE AREA	WIDTH(m.)		36.6		LENGTH (m.)		4.57							
JULIAN DAY	day 228, 1990		day 333, 1990		day 88, 1991		day 170, 1991		day 226, 1991		day 262, 1991		TOTALS (without day 228, 1990)	
SLOPE (%)	6.3		6.3		6.3		5.2		5.2		6.3			
PRECIP. (mm)	71.6		25.4		0.76?		72.4		41.9		39.4			
LOAD (cu.ft./ft.))	12.93		5.99		3.42		5.21		6.73		5.01			
RUNOFF (mm.)	32.8		15.2		8.7		13.2		17.1		12.7		66.9	
RUNOFF (l.)	5488		2543		1450		2210		2858		2128		11189	
INPUT LOADS (g)														
TSS	45492		8057		5604		25209		3835		3199		45904	
NH4	1.2		1.9		1.9		0.0		0.0		0.0		3.8	
NO3	20.3		1.2		0.4		2.3		2.5		2.9		9.3	
ON	71.0		9.4		3.1		27.1		4.4		3.8		47.8	
SP	3.7		1.2		0.6		0.2		0.6		2.3		4.9	
PP	23.70		2.30		1.50		12.70		1.70		0.80		19.00	
OUTPUT LOADS (g)														
WIIDTH(m.)	4.2	8.4	4.2	8.4	4.2	8.4	4.2	8.4	4.2	8.4	4.2	8.4	4.2	8.4
RUNOFF (mm.)	31.2	31.9	16.1	13.9	10.8	6.5	5.0	8.1	7.5	1.1	6.0	1.8	45.4	31.5
RUNOFF (l.)	5822	6558	3008	2860	2009	1328	930	1673	1392	236	1123	375	8462	6472
TSS	25,490	18,905	2,908	4,616	1,066	2,550	1,137	1,307	170	0	536	336	5,817	8,809
NH4	0.39	0.09	1.90	1.80	4.30	2.00	0.01	0.01	0.01	0.01	0.01	1.80	6.23	3.82
NO3	11.60	10.40	1.60	1.30	2.20	0.60	3.00	3.10	1.40	0.01	5.20	0.01	13.40	5.02
ON	43.21	34.61	10.30	8.30	8.40	0.70	4.09	2.69	1.59	-	2.99	-1.79	27.37	11.69
SP	2.80	2.70	1.80	2.10	2.60	1.40	1.00	2.20	0.90	0.01	3.60	1.10	9.90	5.71
PP	21.20	17.90	2.30	1.20	1.60	-	1.90	0.60	0.30	-	0.40	-0.30	6.50	1.80
INPUT CONCENTRATION (mg/l)														
TSS	8289		3168		3865		11407		1342		1503		4103	
NH4	0.22		0.75		1.31		0.00		0.00		0.01		0.34	
NO3	3.70		0.47		0.28		1.04		0.87		1.36		0.83	
ON	12.94		3.70		2.14		12.26		1.54		1.78		4.27	
SP	0.67		0.47		0.41		0.09		0.21		1.08		0.44	
PP	4.32		0.90		1.03		5.75		0.59		0.38		1.70	
OUTPUT CONCENTRATION (mg/l)														
TSS	4378	2883	967	1614	531	1920	1223	781	122	0	477	896	687	1361
NH4	0.07	*0.01*	0.63	0.63	2.14	1.51	0.01	0.01	0.01	0.04	0.01		0.74	.59
NO3	1.99	1.59	0.53	0.45	1.10	0.45	3.23	1.85	1.01	*0.04*	4.63		1.58	0.78
ON	7.42	5.28	3.42	2.90	4.18	*0.53*	4.40	1.61	1.14	0.00	2.66		3.23	1.81
SP	0.48	0.41	0.60	0.73	1.29	1.05	1.08	1.32	0.65	*0.04*	3.21		1.17	0.88
PP	3.64	2.73	0.76	0.42	0.80	0.00	2.04	0.36	*0.22*	0.00	0.36		0.77	0.28
PERCENT REMOVAL														
TSS	47%	65%	69%	49%	86%	50%	89%	*93%*	91%	100%	68%	40%	60%	61%
NH4	69%	*94%*	15%	16%	-63%	-15%	-138%	-32%	-105%		5%		-53%	-45%
NO3	46%	*57%*	-13%	4%	-297%	-64%	-210%	-78%	-15%	95%	-240%		1%	33%
ON	43%	59%	7%	21%	-96%	75%	64%	*87%*	26%		-50%		31%	52%
SP	29%	*39%*	-27%	-56%	-213%	-155%	-1088	-1353	-208%		-197%		-72%	-42%
PP	16%	37%	15%	54%	23%	*100%*	64%	94%	64%		5%		24%	42%

In the case of ammonia, this can be attributed to the low average input concentration of 0.34 mg/l, as well as the nutrient transformations in the filter strip as discussed above. The losses of soluble P suggest a similar mechanism. These authors also observed the accumulation of sediment in the first meter of the filter strip. However, note that there were minimal, or even negative, reductions of TSS by the longer filter strip, as runoff loads and EMCs from several storms actually seemed to increase from the longer strip. This result of several events suggests that the filter strip cover may have been less than optimal. It should be noted that the total loads decreased due to reductions in runoff volumes.

Recently, Barett and others (1997) reported results from a study of runoff filtration by median strips in Texas. In this study of two medians, roadway loads were measured from road inlets or bridge scuppers adjacent to the median, and median strip EMCs were measured from storm drains that drained the medians. Respective means of TSS, nitrate, TKN, total P and zinc EMCs from the roadways were 157, 0.91, 2.17, 0.55, and 0.347 mg/l for the U.S. 183 site, and 190, 1.27, 2.61, 0.24, and 0.129 mg/l at the Walnut Creek site. Respective means of TSS, nitrate, TKN, total P and zinc EMCs from the median strips were 21, 0.46, 1.46, 0.31, and 0.032 mg/l for the U.S. 183 site, and 29, 0.97, 1.45, 0.16, and 0.032 mg/l at the Walnut Creek site.

The authors noted that flow concentrations did not decrease substantially as flow traveled down the median to the drain, indicating that nearly all of the reduction occurred in the side slopes. The side slopes ranged from 7.5 to 8.2 meters wide, with slopes from 7.2% to 9.4%. According to their estimate of annual loads using the “Simple Method” to estimate runoff volumes, minimal infiltration occurred in the C and D soils in the medians. As a result, filtration was the main process involved in reducing the loads. The authors also observed the accumulation of sediments in the first several feet next to the roadway, with accumulated sediment developing a lip at the edge of the pavement.

While these reductions were substantial, inspection of the experimental design as described in Walsh and others (1997) reveals that these EMCs represent the average (central tendency) of 34 storm events, and a paired analysis was not used. (Several storms monitored for roadway runoff were not monitored for median runoff, and vice versa.) Therefore, these results do not represent a stringent input/output relationship. Furthermore, median flows included flows from other larger pervious areas on the Walnut Creek site, and only half the median of the U.S. 183 site intercepted roadway runoff. At best, the monitored median flow thus includes at least a 50% contribution by overland flow that is not affected by roadway runoff. This predominance by pervious surfaces may explain the increase in fecal coliform observed in the swale runoff.

Even though similar results were obtained for median strip EMCs, it is difficult to ascertain what the EMCs would be at the toe of the median strip itself from this design. Using the median grab samples in the U.S. 183 site as a guide, comparison with the reported EMCs suggests that toeslope TSS EMCs would be roughly twice that reported. To resolve the input/output issue, results from certain storm events are summarized in Table 9-3. To obtain average EMC over the events, EMCs were multiplied by the event runoff volumes, summed and then divided by the sum of event volumes. Table 9-3 provides a better illustration of filter strip efficiencies because it represents the flow-weighted averages from cumulative mass loads.

Using this flow-weighted approach, respective means of TSS, nitrate, TKN, total P and zinc EMCs from the roadways were 335, 1.35, 1.70, 0.35, and 0.268 mg/l for the U.S. 183 site, and 185, 1.27, 1.72, 0.22, and 0.121 mg/l at the Walnut Creek site. Respective means of TSS, nitrate, TKN, total P, and zinc EMCs from the median strips were 13.7, 0.70, 1.04, 0.288, and 0.031 mg/l for the U.S. 183 site, and 17.8, 0.74, 1.04, 0.155, and 0.037 mg/l at the Walnut Creek site. Respective reduction of TSS, nitrate, TKN, total P, and zinc EMCs from the roadways were 96%, 48%, 39%, 17%, and 88% for the U.S. 183 site, and 90%, 42%, 40%, 29%, and 69% at the Walnut Creek site. However, it should be emphasized that these figures represent the upper end of efficiency ratios due to the masking effect of unloaded pervious runoff. Using 5 lanes (60 feet) of roadway, the hydraulic loading rate for a 1.5 inch event would be 7.5 cu. ft. per linear foot of filter length.

Efficiency ratios have also been reported for a filter strip system installed for a ten acre shopping center in Virginia (Yu and others, as cited in CWP, 1994). In this system, 0.4 watershed inches of runoff were diverted to 600 foot long level spreader discharging into a 150 foot wide filter strip with a 6% slope. Efficiency ratios in the upper 75 feet were relatively poor, since vegetative cover was sparse, and gullies had formed. Efficiency ratios at the bottom were similar to that reported by Barrett and others (1998). It is instructive to note that hydraulic loading rates for this filter strip are 24.2 cu. ft. per linear foot of filter length. This loading rate is over 3 times that of the typical median strip situation, and nearly twice of the highest reported by Dillaha and others (1989), so the poor results in the first 75 feet may not be unexpected.

Woodard and Rock (1995) reported the extent of filter strip reduction in TSS and Total P EMCs from homesite construction site runoff in Maine. This study reported substantial reductions of these pollutants over forested filter strip widths of 15 meters, while shorter strips were less effective. The longer filter width needed was attributed to the fairly sparse surface cover of the filter. Slope effects were manifest more in terms of increased loading EMCs than by decreased efficiency ratios. Post-construction loadings were much lower, and the filter strip reductions were proportionately less. Total phosphorus EMCs leaving the filter were fairly high, reflecting the background phosphorus contributions from the forest canopy.

Table 9-3: Input and Output Volumes and EMCs from Highway Median Strips
Source, Barrett and others, 1997

AREA	1200	%IMP.	IMP AREA (sq.m.)		1200	IMP. LENGTH (m)		37.3	WALNUT CREEK SITE					
RV	0.95	100%	PER. AREA		0	IMP WIDTH (m.)		32.2						
STORM	6	15	16	19	20	26	27	28	30	32	31	34	ALL EVENTS	
EMCS FROM ROAD													WEIGHTED AVERAGE	
TSS	257	104	93	26	23	54	98	227	147	256	526	113	185.44	
NO3	0.67	4.74	4.12	1.16	0.45	0.95	1.01	1.21	1.14	1.00		3.37	1.27	
TKN	1.22	3.20	2.14	1.59	3.04	0.81	2.05	1.79	1.73	1.95	1.85	3.00	1.72	
TP	0.33	0.23	0.28	0.48	0.21	0.07	0.16		0.15	0.35	0.39	0.25	0.219	
ZN	0.178	0.110	0.076	0.036	0.024	0.007	0.280	0.085	0.093	0.131	0.280	0.226	0.121	
SWALE OUTPUT														
AREA	104600	%IMP.	IMP AREA (sq.m.)		39352	IMP WIDTH (m)		37	SWALE WIDTH		18	%SWALE		
RV	0.23	38%	PER. AREA		65249	IMP LENGTH (m.)		1055	SWALE AREA		18990	0.29	ALL EVENTS	
EMCS FROM SWALE													WEIGHTED AVERAGE & % REDUCED	
TSS	51	24	41	6	4	17	16	8	14	5	60	13	17.8	90%
NO3	1.07	3.69	2.53	3.49	*0.36*	0.50	1.03	0.51	1.45	0.83	0.01	1.22	0.74	42%
TKN	0.90	2.74	2.11	2.11	0.92	0.69	0.97	0.90	0.98	1.01	1.42	1.30	1.04	40%
TP	0.04	0.18	0.22	0.21	0.19	0.19	0.13		*0.11*	0.16	0.15	0.16	0.155	29%
ZN	0.023	0.003	*0.002*	0.002	0.003	0.003	0.018	0.003	0.044	0.122	0.067	0.147	0.037	69%

AREA (sq.m.)	850	%IMP.	IMP AREA (sq.m.)		850	IMP. WIDTH (m)		19	183 SITE			
RV	0.95	100%	PER. AREA		0	IMP LENGTH (m.)		44.7				
STORM NO.	15	16	19	20	22	25	28	32	33	36	ALL EVENTS	
RUNOFF	6800	18480	5940	15260	32320	30600	55180	20490	12360	87389	284819	
EMCS FROM ROAD											WEIGHTED AVERAGE	
TSS	247	117	31	17	135	81	98	3328	522	48	332.9	
NO3	3.29	5.66	2.66	0.8	2.25	0.53	0.55	0.43	1.63	0.94	1.35	
TKN	5.92	1.87	2.99	1.20	2.21	0.62	0.89	2.06	0.31	2.18	1.70	
TP	0.60	0.35	0.51	0.20	0.39	0.16		0.58	0.69	0.30	0.35	
ZN	0.459	0.285	0.279	0.030	0.123	0.126	0.093	0.440	0.690	0.410	0.268	
SWALE												
AREA (sq.m.)	13000	%IMP.	IMP AREA		6764	IMP. WIDTH (m)		19	SWALE WIDTH		17.5	%
RV	0.37	52%	PERV. AREA		6236	IMP LENGTH (m.)		356.0	SWALE AREA		6230	100%
EMCS FROM SWALE											WEIGHTED AVERAGE	
TSS	38	50	3	5	7	14	7	6	6	19	13.7	
NO3	2.71	3.71	0.31	0.20	1.32	*0.20*	0.25	0.41	0.68	0.48	0.70	
TKN	1.97	1.73	1.83	1.18	0.90	0.78	0.63	1.33	1.07	1.12	1.04	
TP	0.46	0.24	0.35	0.21	0.32	0.43			0.28	0.19	0.29	
ZN	0.002	0.003	0.002	0.002	0.002	0.025	0.022	0.070	0.050	0.070	0.031	
PER CENT CONCENTRATION REDUCTIONS												
TSS	85%	57%	90%	71%	95%	83%	93%	100%	99%	60%	96%	
NO3	18%	34%	88%	75%	41%	62%	55%	5%	58%	49%	48%	
TKN	67%	7%	39%	2%	59%	-26%	29%	35%	-246%	48%	39%	
TP	23%	31%	31%	-5%	18%	-169%			59%	37%	17%	
ZN	100%	99%	99%	93%	98%	80%	76%	84%	93%	83%	88%	

9.2 VFS-MOD MODEL

A decade ago, Munoz-Carpena, Parsons and Gilliam (1992) developed the VFS-MOD filter strip model. This model uses the *CN* method to predict runoff, and the Modified Universal Soil Loss Equation (MUSLE) to predict sediment loads from the source areas. Filter strip runoff volume reductions are computed by the Green-Ampt equation, while flow through the strip is based upon a kinematic wave approximation. Sediment reductions are computed according to the procedures developed in the 1970's at the University of Kentucky by the team of Tollner, Barfield and Hayes (See Munoz-Carpena and others (1992) for references). This model explicitly computes the runoff volume and sediment losses as it passes through the strip. It also accounts for the development of the initial wedge of sediment that occurs in the first few feet of the strip discussed above.

The VFS-MOD model was subsequently validated in studies of 27 rainfall events in the North Carolina Piedmont (Munoz-Carpena and others, 1999). These investigators reported that the hydrologic responses, sediment loads and subsequent reductions predicted by VFS-MOD were very close to observations, so long as no channelization occurred. Table 9-4 shows how the filter strip performance closely matched predicted observations, with an average error below 6 percent. Note that most of the load reductions were close to the concentration reductions, suggesting that decreases in runoff volume were not substantial.

Note that the 8.4 meter wide strip was much more effective than the 4.2 meter strip at loads above 7,900 mg/l. However, such loads are extremely high for urban situations. At loads below 2,500 mg/l more typical of urban runoff, removal efficiencies in the 4.2 meter wide strips were quite good. This is in accordance with the results of Barrett and others (1997) displayed in Table 9-3. This further confirms that even narrow filter strips can provide good removal rates for urban runoff.

Table 9-4: Input/Output Loads and EMCs from Filter Strips
Source, Munoz-Carpena and others (1999)

Strip Event	112-92 g4	112-92 r1	151b-92 g4	331a-92 g4	331a-92 g8	024-93 g4
Strip Width (m)	4.2	4.2	4.2	4.2	8.4	4.2
INPUT						
Concentration (mg/l)	1080	750	2440	7930	7930	11470
Sediment Load (g)	287.0	188.9	968.3	5788.0	5788.0	6187.8
OUTPUT						
Runoff Volume (m3)	0.1873	0.1129	0.1905	0.6411	0.5240	0.3687
% Model Error	-10.6%	23.5%	-1.6%	-1.1%	5.3%	12.1%
Sediment Load (g)	30.5	17.9	20.1	2488.0	345.0	639.0
% Model Error	1.3%	9.2%	0.7%	0.4%	19.5%	-3.5%
Concentration (mg/l)	162.8	158.5	105.5	3880.8	658.4	1733.1
Conc. Reduction	84.9%	78.9%	95.7%	51.1%	91.7%	84.9%
Load Reduction	89.4%	90.5%	97.9%	57.0%	94.0%	89.7%

In Canada, Abu-Zreig and others (2001), recently confirmed the validity of the VFS-MOD model, once concentrated flows across the filter strip were explicitly modeled as discrete segments. Coefficients of determination (R^2) for infiltration volume and trap efficiency were 0.95 and 0.90, respectively, and the results were highly significant (p<0.01). These results confirm that the VFS-MOD model is an effective method to predict filter strip removal kinetics.

9.3 FILTER STRIP REMOVAL KINETICS

The preceding discussion highlights three factors that affect the performance of filter strip BMPs: filter strip width, slope, and hydraulic loading rate, assuming a uniform turf cover condition. There is an abundant literature demonstrating that filter strip trap efficiency increases as filter strip width increases (Dillaha and others, 1989; and many others), although there are occasional circumstances where sediment concentrations increase as filter strip widths increase (Parsons and others, 1993). The authors attributed this latter finding to the relatively poor cover in the filter strip. It should be noted that most TSS deposition occurs within the first few feet, and trapping efficiencies are statistically identical from 15 to 30 feet (Mendez and others, 1989).

To develop a relationship between filter strip width and trap efficiency, Abu-Zreig (2001) ran VFS-MOD on filter strips of varying widths and slopes. Figure 9-1 displays how predicted trap efficiencies approached the maximum reported removal of 93 percent in filter strip widths of 40 to 50 feet. The hydraulic loading rate was 10.8 cubic feet per foot, a value typical for urban runoff loading rates. However, the input concentration was a fairly high 4,000 mg/l, a value roughly 20 times that expected in urban runoff. Note how the VFS-MOD results generally correspond to the observations of Dillaha and others (1989). A relationship following the asymptotic form of equation (32) is applicable to these results. Excluding hydraulic loading rate adjustments, the form of equation used in DURMM is shown in thicker lines in Figure 9-1.

Abu-Zreig (2001) also presented the trap efficiency for the clay fraction, which has much lower efficiencies than suspended sediment as a whole. This is not surprising, since clays have a much slower settling kinetics for a given filter surface area or detention time. Unlike the exponential decay seen in coarser sediments, the trapping efficiency of clay shows nearly linear removal kinetics. Since adsorbed phosphorus is preferentially bound to the clay fraction, this has implications for particulate phosphorus removal.

A counterintuitive finding of this study is that the effect of slope is much less pronounced, and it is absent if the strip is wide enough. In essence, the effect of increasing slope is to shift the x-intercept to the right, so its effect is more pronounced at lesser widths. This relationship is partially due to the fact that increasing slope increases Manning's roughness value under submerged flow conditions (Kao and Barfield, 1978; Kouwen and Lee, 1980). This finding was confirmed by the observations of Abu-Zreig and others (2001). Since both the numerator and denominator of Manning's equation thus increase simultaneously, flow velocity and filter strip retention time is comparatively unaffected. As a result, it is not surprising that the 18% increase in trap efficiency between 11% and 16% slopes shown in Table 9-1 is much less than the 45% reduction in slope. Therefore, slope effects in filter strip efficiency seem to be fairly minor, and have minimal effect on maximum efficiency ratios.

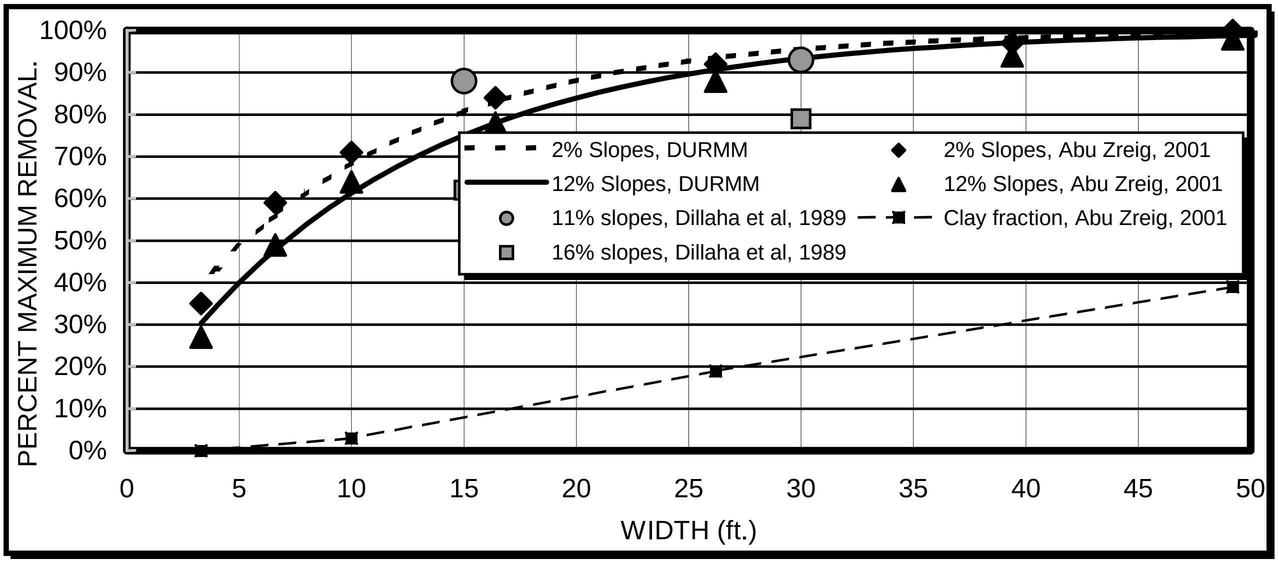

Figure 9-1: Percent Of Maximum Trap Efficiency as a Function of Filter Strip Width and Slope
Sources: Dillaha and others, 1989; Abu-Zreig, 2001

Given the validity of the VFS-MOD model, the following discussion presents model results for a turf grass filter at varying slopes and hydraulic loads. Input parameters were a silty-clay filter strip soil at a moisture deficit of 17.5 percent, suspended sediment with a fine particle distribution (d_{50} = 23 microns), an input concentration of 300 mg/l, and a hydraulic load of 10.5 cubic feet per foot. Results for varying slopes and widths are displayed in Figure 9-2.

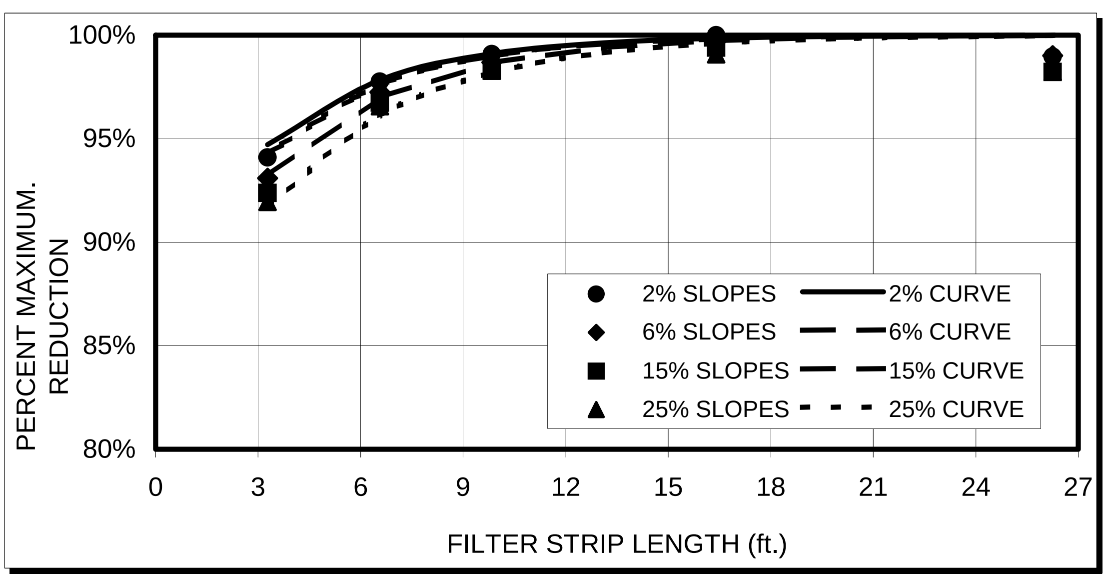

Figure 9-2: Percent Of Maximum Reduction Efficiency as a Function of
Filter Strip Width and Slope

Even under these less than optimal conditions, with silty-clay sediment and tight, moist soils, the computed maximum reduction efficiency was 96.9 percent, a value similar to that reported by Barrett and others (1997). There was very little change in this value (±0.3 percent) for loading rates of 200 and 100 mg/l on a 5 meter strip, so results for these lower loading rates were not computed further.

These model runs further demonstrate the relative lack of sensitivity to slope, even for slopes as high as 25 percent. Note that the scale has been exaggerated to highlight the small differences in removal efficiencies. When removal efficiency is this high, a small difference in removal efficiency represents a large difference in the final output concentration. Therefore, the computation of such subtle differences is important in predicting output loads from filter strips. Note also that the best performance of the filter strip occurs at a width of 16.5 feet (5 meters), and reduction efficiency decreases somewhat at higher widths. This is due to the model computing volumes that decrease more than the loads at the higher width, resulting in higher concentrations.

This effect of differing hydraulic load has not been explicitly reported in the literature, so additional runs of VFS-MOD were preformed to investigate this aspect of filter strip performance. Using the same input parameters as in Figure 9-2 for a 2 percent slope filter strip, Figure 9-3 presents the effects of increasing the hydraulic loads to 21.0 and 31.5 cubic feet per foot of filter strip length:

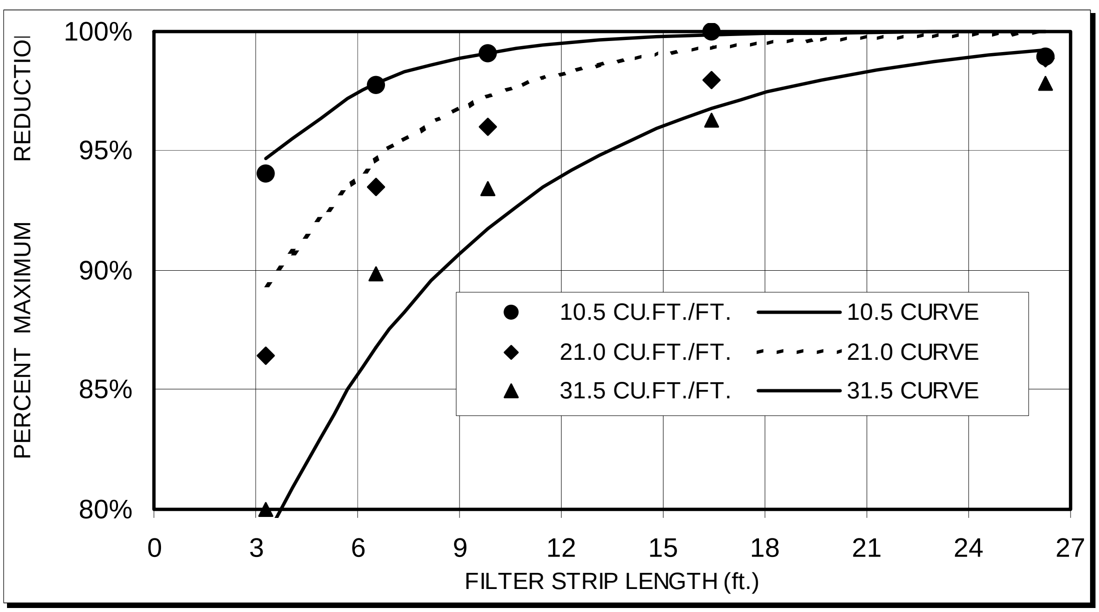

Figure 9-3: Percent Of Maximum Reduction Efficiency as a Function of Filter Strip Width and Hydraulic Load

Over the range of expected loading rates, variations in hydraulic load are seen to have a much greater effect than is projected for slopes. As in the case of slopes, these effects become less prominent as the filter strip gets wider.

Another important variable is the stem density. The results presented above assume that the filter strip cover is in good condition. Obviously, such results would not be expected for filter strips in poor conditions or in winter, when the effective stem density will be less. Since this is the season when most runoff occurs, results for poorer conditions and grasses other than turf need to be examined. In VFS-MOD, bluegrass turf is allocated as having a stem distance (density) of 1.65 cm between stems, while mixed grasses are allocated as having a density of 2.15 cm. (Grasses with higher densities are not recommended in the documentation.) Under the same input parameters as Figure 9-1, the results for the mixed grass stem density are shown below in Figure 9-4. Surprisingly, increasing the stem density by a factor of nearly three to 6.0 did not seem to materially affect performance.

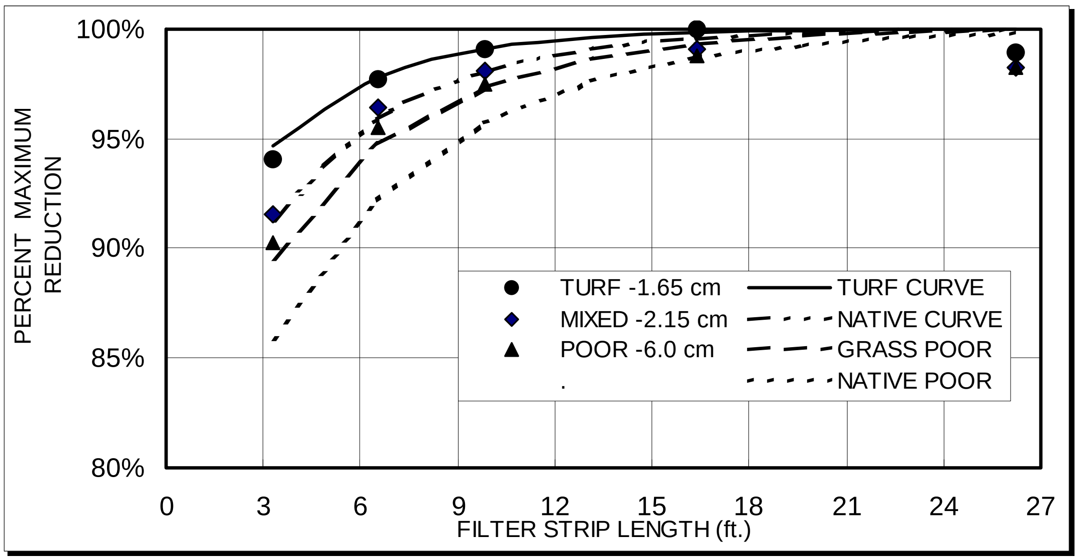

Figure 9-4: Percent Of Maximum Reduction Efficiency as a Function of Filter Strip Width and Stem Density

To replicate these trends with a minimum of computational overhead, DURMM uses the following expression and coefficient values to relate maximum design efficiency ratio, $R_{max}\%$, to maximum possible efficiency ratio *Max%*, coefficient *K*, width *w*, hydraulic loading rate *r*, slope *s*, and stem density factor *d*:

$$R\%_{\max} = Max\% \times \left\{1 - e^{f(K,w,r,s,d)}\right\} \quad (34)$$

where

$$f(K,w,r,s,d) = K \times (A - w) \times (1 - r/B) \times (1 - C/s) \times 1/d \quad (35)$$

Coefficients for (35) are listed below:

K	A	B	C
0.340	-2.30	55	0.65

Stem density factors are listed below:

TURF GOOD	MIXED GOOD	TURF POOR	NATIVE POOR
1.00	1.20	1.30	1.50

The curves derived from equations (34) and (35) are shown in Figures 9-2 through 9-4. While the correspondence to VFS-MOD results is not perfectly matched by these curves, the general trends and range of values are quite satisfactory. In this manner, DURMM provides the capability to rapidly determine the most important elements of filter strip performance in relation to specific site design parameters. It is recommended that poor conditions be applied to the design to account for variability in seasonal growth and potentially poor maintenance. To account for the worst circumstances, an estimated curve for native grasses in poor condition is shown in Figure 9-4.

9.4 MAXIMUM EFFICIENCY RATIOS AND IRREDUCIBLE CONCENTRATIONS

Table 9-5 summarizes data in Tables 9-1, 9-2, and 9-3 relating to the maximum likely efficiency ratios and minimum effluent EMCs for which grass filter strips are capable. The lowest values and highest removal rates are outlined in italics in Tables 9-1 through 9-3. Since the literature is so sparse to refine some of these values, reference is also made to the literature values for biofiltration swales discussed in the next Chapter. Note that the minimum values and maximum removal are intended to be representative values, not the most extreme value observed, which may reflect sampling error.

Based upon the data, particulate and adsorbable soluble species such as TSS, organic N, PP, ammonia and zinc show high potential maximum efficiency ratios in the range of 60% to 97%. Ammonia is subject to both processes of transformation, adsorption and dilution, with projected reductions based upon the Texas results. Although total P showed lower efficiency ratios in Barrett and others (1997), this may be due to the relatively low inflow concentrations. There were high efficiency ratios for particulate P in Dillaha and others (1989) and Parsons and others (1993), where concentrations were much higher. However, recent papers by Rudra and others (2001a, 2001b) suggest that particulate phosphorus removal is likely to be substantially less than TSS removal, since phosphorus is preferentially adsorbed to the finer particles, which have much lower filtration efficiency. Since this depends on both the characteristics of incoming runoff and different filter strip length relationships, no attempt has been made in this version to precisely address the differing reduction kinetics. However, note that the maximum removal percentage for particulate phosphorus is allocated at a conservative 60 percent.

Even though the reduction in nitrate EMCs seems to mostly reflect dilution effects, this is modeled as a reduction in EMCs since DURMM accounts for overall event hydrologic processes, and infiltrated nitrate is largely sequestered in the root zone. However, atmospheric deposition requires that the minimum concentration for nitrate be in the range of 0.15 mg/l, even though reported effluent values are as low as 0.04 mg/l. Since there are many cases where efficiency ratios are negative, the maximum efficiency is projected at 40 percent. The maximum soluble P efficiency ratio is even lower, since biological transformations in the filter strip often exceed input loads due to release of previously sequestered soluble P. Since there is no data on copper removal in the filter strip literature, results from the bioswale analysis in Chapter 10 are projected to apply to filter strips.

To obtain the efficiency ratio for Filter Strip BMPs, maximum reduction percentage *Max%* from Table 9-5 is entered as the first term of the filter strip design equation (34). The rest of the terms in equation (35) are set by the actual design parameters to obtain design efficiency ratio $R_{max}\%$, expressed as a percent of the potential maximum ratio for each pollutant of interest. The minimum EMC values entered as part of the general efficiency equation (32). Actual efficiency ratio *R%* is then depends upon the input concentration entered into (32).

Table 9-5: Maximum Efficiency Ratios and Minimum EMCs for Filter Strips

POLLUTANT	TSS	PP	SP	ON	NH_4	NO_3	Cu	Zn
MAXIMUM REDUCTION %	97%	60%	15%	90%	70%	40%	85%	97%
MINIMUM EMC (mg/l)	5	0.10	0.05	0.40	0.15	0.15	0.002	0.006

The BMP design module of DURMM described in Appendix A provides estimates for filter strip performance as a function of loading rate, pollutant levels, width, slope, and stem density. Design standards, construction specifications and details of the filter strips and level spreader design are set forth the Green Technology Standards, Specifications and Details, Appendix B.

9.5 FILTER STRIP BMPs AND INTEGRATION WITH TERRACE BMPs

An imperative feature in the design in all sheet flow filtration BMPs is the provision of sheet flow conditions. Filter strips are thus best suited for situations where runoff has not been concentrated, as is found from parking lots, driveways, sidewalks, and uncurbed streets. When used as median strips or adjacent to parking lots, the paving itself is an effective level spreading device. However, if filter strips are used after flow has become channelized, their efficiency ratios are not nearly as effective (Dillaha and others, 1989, and many others).

Therefore, an engineered level spreader is necessary to restore sheet flow discharge to the filter strip. Absolutely level surfaces are necessary, and the appropriate structure must be provided. Where runoff from rooftops and curbed roadways is conveyed through pipes, a level spreader is necessary in order to reestablish sheet flow conditions needed for filter strips. See the level spreader details set forth in the Standards, Specifications and Details in Appendix B. Since biofiltration swales are as effective as filter strips in these circumstances, it is questionable whether the potentially less reliable performance of level spreaders is worthwhile, unless there are special circumstances, such as concentrated discharge directly onto a hill slope where a biofiltration swale would require excessive grading.

Filter strips are an integral part of terrace BMPs. In this case, the side slope entering the terrace operates as a filter strip, functioning in a manner analogous to the median filter strips reported in Texas (Barrett and others, 1998). Filtration reductions in the conveyance channel are assumed to be minimal, as flow depths are relatively deep, and runoff is already filtered. Since the side slope provides such substantial filtering, further reductions along the channel would provide minimal additional removal. DURMM thus allocates terrace side slope filtering according to the design parameters in equations (34) and (35).

CHAPTER 10 BIOFILTRATION BMPs

10.1 BIOFILTRATION SWALE RESULTS

Biofiltration swales represent a class of BMPs in which filtering occurs as flow travels along a defined channel. The filter media is usually grass, although ground cover vines, sedges and rushes can also be used. Channel flow velocities along biofiltration swales would seem to be faster than sheet flow through filter strips, since flow depths are greater. However, this may not be the case, as discussed in Section 5.6. Typical flow depths are deeper than sheet flow BMPs, although they should be well below the top of vegetation in properly designed swales. As discussed in Chapter 6, it is very important that vegetation not be submerged, since this causes the vegetation to bend over with the flow, which reduces roughness. Resulting flow velocities will be much greater, and the opportunity for contact filtering is less.

Biofiltration swales are well suited for concentrated flow situations, where runoff has already been collected by piped conveyance systems. However, they are also very effective as a conveyance system in themselves, they can also be designed to provide detention to meet the discharge criteria set forth in Chapter 4. Using detention routing in a similar manner, terraces represent a swale that is surcharged laterally, instead of from a point discharge. As such, terrace BMP removal processes could be considered analogous to that involved in the median filter strips discussed in the previous Chapter.

The literature on biofiltration swale EMC reductions is rather sparse. While there is some data from Florida (Yousef and others, as cited in CWP, 1995), the data is expressed in terms of mass reduction where considerable infiltration had occurred, so removals due to EMC reductions are elusive. The only data set from a swale where infiltration was minimal is presented in a report by the SWPCD (1992). This report measured the responses to 12 storms of a 187 foot long bioswale 5 feet wide with 3:1 side slopes in Seattle, Washington. The authors attempted to ascertain the reduction in swale efficiency if swale retention time was halved by doubling the input flows. However, as discussed in Chapter 6, submerged flow velocities in this swale were essentially constant, regardless of flow rates. As table 5-6 in SWPCD (1992) shows, the cross-sectional areas for the "shorter" runs were identical to the cross-sectional area used in longer runs at similar flow rates, thus the presumed reductions in retention time were based upon a "shorter" effective width. This width was assumed to be half the measured width of 187 feet, since halving the flow was supposed to halve the area. However, the latter is true only in terms of cross-sectional area, not overall wetted area. Table 10-1 presents results from the SWPCD (1992) study.

Flow weighted averages are determined as the sum of products of each event EMC times its flow, divided by cumulative flow of the events measured. Note that Table 10-1 has partitioned total P and the metals into soluble and particulate fractions. Values shown in italics are either below detection limits, flagged as unreliable, or a constituent EMC that had a higher EMC than the total EMC (eg, where dissolved copper EMC was greater than the total copper EMC). These values are excluded from the flow-weighted averages.

Table 10-1: Performance of Biofiltration Swale in Washington State Source: SWPCD, 1992

PARA-METER	INDIVIDUAL STORM EVENT DATA												FLOW WEIGHTED AVERAGES		
DATE	6/20/91	7/15/91	7/24/91	8/9/91	10/24/91	10/31/91	11/17/91	1/16/92	1/23/92	3/27/92	4/17/92	4/29/92	6/20/91 to 1/23/92	3/27/92 to 4/29/92	ALL STORMS
Qavg.(cfs)	0.16	0.02	0.40	0.31	0.21	0.10	0.07	0.03	0.05	0.21	0.25	0.11			
Qpeak(cfs)	0.29	0.12	**0.78**	**1.35**	**0.61**	0.18	0.29	0.09	0.18	**0.49**	**0.65**	**0.56**			
DEPTH(ft.)	0.09	0.01	0.20	0.16	0.11	0.06	0.04	0.02	0.03	0.11	0.13	0.06			
FLOW(cu.ft.)	3168	180	4320	4464	5481	2700	2898	378	1260	2268	3600	2376	2761	916	2758
INPUT CONCENTRATION (mg/l)															
TSS	18	51	180	32	190	19	57	91	51	150	190	150	108	167	113
NO3	0.210	0.640	0.470	0.310	0.230	0.250	0.06	0.590	0.230	0.420	0.031	0.230	0.314	0.195	0.255
SP	0.017	0.043	0.031	0.042	0.088	0.036	0.005	0.005	0.005	0.007	0.022	0.005	0.045	0.013	0.033
PP	0.075	0.287	0.309	0.053	0.142	0.074	0.024	0.115	0.020	0.008	0.138	0.235	0.137	0.130	0.122
Zn-D	0.048	0.150	0.110	0.023	0.021	0.015	0.003	0.003	0.002	*0.042*	0.034	0.019	0.043	0.028	0.035
Zn-P	0.020	0.100	0.100	0.048	0.029	0.023	0.013	0.107	0.094	*0.005*	0.096	0.121	0.053	0.106	0.058
Cu-D	0.002	0.021	0.010	*0.007*	*0.002*	*0.004*	*0.021*	0.002	0.001	0.001	0.002	0.002	0.009	0.002	0.004
Cu-P	0.000	0.010	0.003					0.006	0.005	0.006	0.010	0.012	0.004	0.009	0.006
OUTPUT CONCENTRATION (mg/l)															
TSS	2	12	14	**34**	6	4	4	7	9	**140**	**50**	**150**	13	104	35
NO3	0.210	2.100	1.200	0.570	0.310	0.210	0.060	0.740	0.240	0.490	0.059	0.250	0.544	0.233	0.414
SP	0.014	0.043	0.027	0.088	0.051	0.016	0.005	0.005	0.005	0.005	0.013	0.034	0.042	0.017	0.032
PP	0.056	0.207	0.213	0.022	0.045	0.036	0.010	0.026	0.010	0.000	0.117	0.146	0.075	0.093	0.072
Zn-D	0.010	*0.072*	0.055	*0.058*	0.007	0.002	0.014	0.002	0.001	*0.024*	0.031	0.023	0.021	0.028	0.020
Zn-P	0.022		0.018		0.011	0.004	0.008	0.047	0.012	*0.054*	0.051	0.087	0.016	0.065	0.025
Cu-D	*0.001*	0.012	*0.013*	0.001	*0.004*	*0.005*	*0.006*	0.001	0.001	0.001	0.002	0.002	0.001	0.002	0.002
Cu-P		0.000		0.001				0.002	0.001	0.014	0.007	0.016	0.001	0.012	0.007
EMC REDUCTION															
TSS	89%	76%	92%	-6%	97%	79%	93%	92%	82%	7%	74%	0%	88%	38%	69%
NO3	0%	-228%	-155%	-84%	-35%	16%	0%	-25%	-4%	-17%	-90%	-9%	-73%	-19%	-63%
SP	18%	0%	13%	-110%	42%	56%	0%	0%	0%	29%	41%	-580%	8%	-30%	4%
PP	25%	28%	31%	58%	68%	51%	58%	77%	50%	100%	15%	38%	45%	28%	41%
Zn-D	79%	52%	50%	-152%	67%	87%	-367%	33%	50%	*43%*	9%	-21%	51%	1%	44%
Zn-P	-10%	100%	82%	100%	62%	83%	38%	56%	87%	*-980%*	47%	28%	69%	38%	56%
Cu-D	*50%*	43%	*-30%*	*86%*	*-100%*	*-25%*	*71%*	50%	0%	0%	0%	0%	85%	0%	61%
Cu-P		100%						67%	80%	-133%	30%	-33%	67%	-22%	-21%

Note how the bioswale generally performed well in terms of TSS reductions until the last three storm events. Comparison of the weighted averages displays very different responses from the other storm events in terms of EMC efficiency ratios, some of which can be attributed to reductions in input concentrations. However, even though input TSS EMCs were generally similar, the poor performance of the 3/27/92 and 4/29/92 storms is difficult to explain. The authors noted that the storms in April conveyed substantial silt into the swale, which may have been resuspended during subsequent runoff events.

However, note that poorer results occurred with storms at the higher flow rates, 8/9/91 and 3/27/92. These storms were the worst performing in terms of TSS reduction, suggesting that hydraulic loading rate (or flow depth) has an effect. However, the storms of 7/24/91 and

10/24/91 were among those with the deepest flow depths, yet they obtained excellent reductions. It is possible that these contrary results indicate resuspension of previously deposited fine sediments. It is also important to note that the peak and average flows were quite low in general, especially considering a 14.7 acre watershed with 44% impervious cover. Average flow depths in these storms were quite low, suggesting that performance will be compromised if flow depths get at all close to emergence.

The preceding discussion, however, does not provide any information as to the pertinent details needed to optimize the design of bioswales. The effect of relevant parameters such as hydraulic loading rate (or flow depth) and retention time cannot be determined from the SWPCD (1992) study. However, in a companion study to the median strip results discussed above, Walsh and others (1997) measured EMCs in an artificial flume 30 inches wide and 40 meters long at a slope of 0.44%. It was planted in buffalo grass over a 6 inch topsoil and 3 inch gravel matrix with an underdrain to catch leachate. A “cocktail” of synthetic urban runoff was delivered to the flume, with measurements of EMCs at 0, 10, 20 30, and 40 meters along the flume. Flows were set at depths of 3, 4, 7.5 and 10 cm. Since the latter depth is well above the proper depth for biofiltration, its results have not been displayed in the following discussion.

Figures 10-1 and 10-2 display the reductions in EMCs and efficiency ratios for TSS and Zinc in this experiment. Reductions ranged from 22% to 66% for total P, from 14% to 47% for TKN and minimal or negative for nitrate, as can be expected from previous analysis of filtering literature. There was little correlation with distance or depth of flow for these constituents. While zinc reduction percentages were greater than observed in Seattle, the minimum EMC was in the range of 0.05 mg/l.

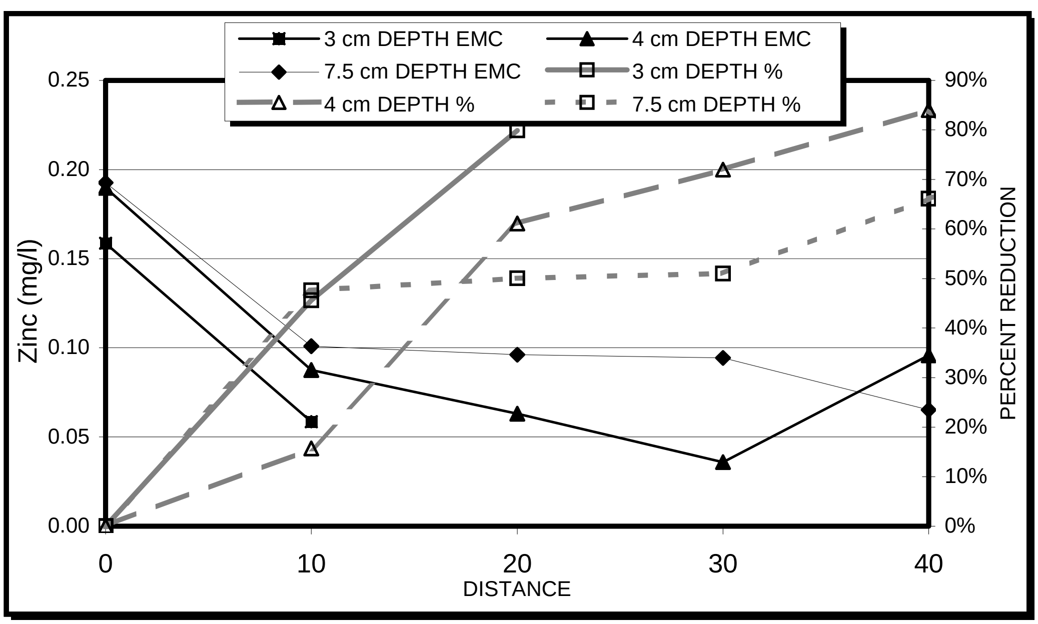

Figure 10-1: Percent Reduction of Zinc as a Function of Distance and Depth
Source: Walsh and others (1997)

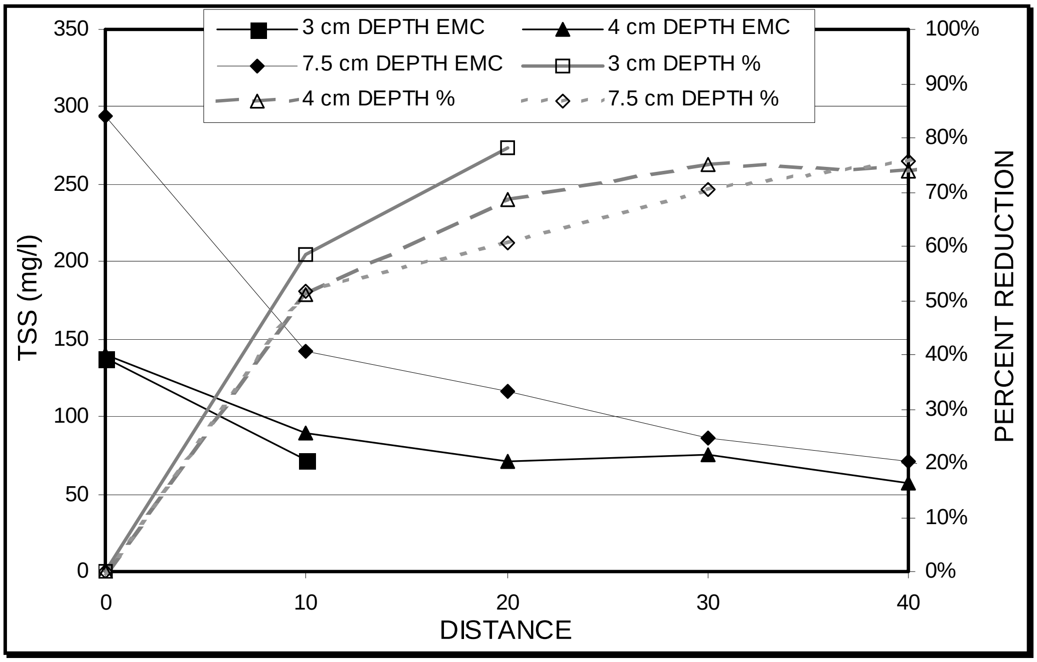

Figure 10-2: Percent Reduction of TSS as a Function of Distance and Depth
Source: Walsh and others (1997)

Note that TSS and Zinc EMCs show a substantial decreasing trend with length. Reductions peaked at around 80% for both these constituents, similar to that observed by the SWPCD (1992) for TSS. Most of the reductions occurred within the first 20 meters, and further losses were relatively minor after 20 meters. Although not shown, the maximum reduction at the 10 cm depth was only 50%, which was reached at 20 meters. Sediment coating on the grass was obvious within the first 3 meters, and still could be observed at 10 meters, but was absent at 20 meters and beyond.

These results suggest that, at the experimental slope of 0.44%, the minimum length for effective TSS and zinc reduction should be at least 20 meters, and preferably 30 meters. Using the geometry of the swale, average flow velocities are calculated between 0.16 and 0.18 ft/sec for the range of flow depths involved. At 30 meters, this suggests a retention time of 9.9 minutes for best results. This value agrees remarkably well with the conclusions of the SWPCD (1992) swale study.

However, note that the final TSS EMC was in the range of 50 to 70 mg/l, roughly ten times the minimum EMC from the Seattle swale. To account for this result, note that the minimum TSS EMC in the leachate was 36 mg/l, with an average of 39 mg/l in the 7.5 cm tests, and 57 mg/l in the 4 cm tests. Given a profile of 6 inches of soil on top of only 3 inches of gravel, a substantial fraction of TSS is lost from the soil profile in every test, so it is not filtered at any length. As such, this fraction seems to be an artifact of the design that may not be applicable to typical biofiltration swale designs for urban runoff.

If the average leachate TSS concentrations are then subtracted from the input and effluent TSS concentrations for the results of the 4 and 7.5 cm tests, the resulting concentrations and efficiency ratios are remarkably similar to that observed in filter strips and in the SWPCD swale.

Figure 10-3 plots the resulting EMCs and reductions as a function of length and retention time. Note that maximum efficiency ratios exceed 90%, and minimum EMCs are 18 and 33 mg/l for the 4 and 7.5 cm tests, respectively. These values are more in line with the results in the SWPCD (1992) swale.

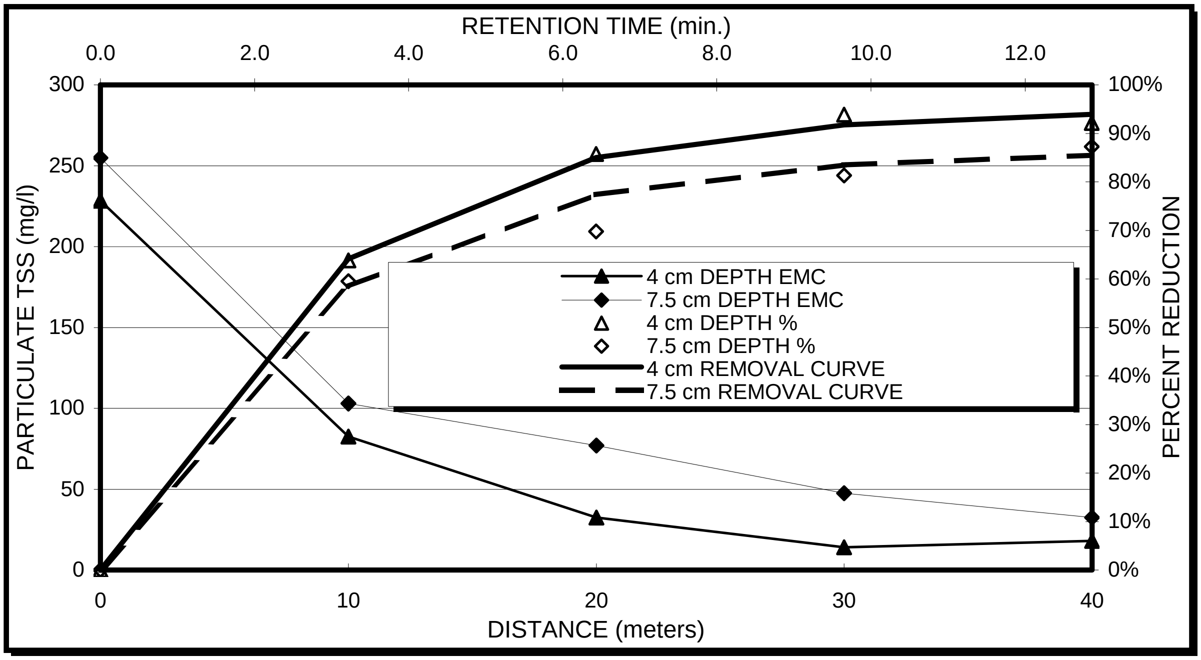

Figure 10-3: TSS EMCs and Reductions as a Function of Distance, Retention Time, and Flow Depth. Data adjusted from Walsh and others (1997)

10.2 BIOFILTRATION SWALE KINETICS

These results show a clear trend for a reduction in efficiency and increase in the minimum concentration with increasing depth of flow, although the minimum EMCs begin to converge by 40 meters. At depths up to 7.5 cm, Equation (36) reproduces these results for efficiency ratio $R_{\max}\%$ as a function of retention time T and depth d (in feet):

$$R_{\max}\% = MAX\% \times (1 - e^{-KT}) \times (A/d)^{B} \quad (36)$$

MAX%	K	A	B
95%	-0.350	0.131	0.15

Using the results of the SWPCD study and this study, Table 10-2 displays the anticipated maximum efficiency ratios and minimum EMCs for bioswales. Again, since the data is rather sparse, reference is made to the filter strip data to project minimum EMCs for certain pollutants, As a result the values are nearly identical to that projected for filter strips, which is to be expected, since similar removal processes by exposure to vegetation are involved.

Table 10-2: Maximum Efficiency ratios and Minimum EMCs

POLLUTANT	TSS	PP	SP	ON	NH4	NO3	CU	ZN
MAXIMUM REDUCTION	95%	95%	45%	90%	70%	45%	85%	85%
MINIMUM EMC (mg/l)	4	0.010	0.005	0.40	0.15	0.15	0.002	0.006

Note that relatively poor reduction (50% compared to 75%) was noted at flow depths of 10 cm (0.33 feet), while good results occur at depths up to 7.5 cm (0.25 feet). For these reasons, biofiltration swales should be designed so that the maximum flow depth does not exceed 0.25 feet during the design flow event, unless a dense uncut stand of native grasses is provided. If swales operate at even lower depths, the performance will be even better.

Since bioswale flow depths need to be totally submerged for decent results, their filtering mechanisms would seem to correspond to that of filter strips. In this light, it is instructive to calculate the retention time in a filter strip, using the sheet flow equation (6), and allocating hydraulic loading as excess precipitation, as is used for impervious disconnection. At a hydraulic loading rate of 12 cu.ft./ft. into a 30 foot filter strip, the resulting excess precipitation would be 0.4 feet. Added to 1.5 inches of rainfall depth, this is a rainfall depth of 7.2 inches. Using equation (6) for a 5% slope at roughness value of 0.45, the travel time through the strip is roughly 4.2 minutes, and 3.1 minutes for a 10% slope. Since these times are at the low end for effective biofiltration, this suggests that either filter strip filtration is more effective for a given time interval (perhaps due to stem density in thatch layer), and/or equation (6), with its parameter values, overstates flow velocities.

10.3 BIOFILTRATION SWALE BMPs

It is essential that the swale soil profile promote vegetative growth, and infiltration, if located above the water table. Therefore, bioswales should be constructed into uncompacted soils where possible. However, construction inevitably involves mass grading, reconfiguration of underlying drainage patterns, and substantial compaction, often deep into native soils. Therefore, prior to topsoil return, the compacted subgrade at the bottom of the swale should be overlaid with a coarse sand mix, which is then subsoiled for several feet, to promote infiltration and biological growth. The facility should then be disked after construction before final grading the topsoil.

The BMP design module of DURMM described in Appendix A provides estimates for biofiltration swale performance as a function of loading rate, pollutant levels, length, width, side slopes, longitudinal slope, and surface cover. Design standards, construction specifications and details of biofiltration swales are set forth the Green Technology Standards, Specifications and Details, Appendix B.

Another important benefit of biofiltration swales is their ability to provide detention storage for events larger than the quality event. By installing stone check dams at regular intervals, a bioswale several feet deep can provide enough storage for reducing the peak flow of even the 100 year event. Check dams constructed with a turf reinforcement matting apron on the downstream side to provide a stable substrate to absorb the energy concentrated in the fall over the dam. Check dams thus absorb the energy of high flows, reducing the potential for resuspension of previously deposited sediments in larger storms. Since the flow from the hydraulic jump on the apron immediately enters the pool created by the next check dam, flow velocities remain quite low throughout, even in extreme events.

Using check dams, many of the benefits of an off-line layout can be provided without the need for flow-splitting devices and the loss of space required for an off-line facility. The BMP design module of DURMM described in Appendix A provides estimates for storage as the check dams fill up. A detail of the typical check dam elevation and profile, including grade drop structures, is displayed in Appendix B.

10.4 INFILTRATING BIOFILTRATION SWALE BMPs

A further important benefit inherent to bioswales is their ability to incorporate an infiltration trench along the center. Since filtration occurs down the sides and along the check dam, by the time runoff flows over the check dam into the stone apron it is already well filtered, so contamination of groundwater by metals is less likely. (However, nitrate loads could still be a problem.) Designing the stone apron as an extension of the infiltration trench, filtered runoff from the upslope section will preferentially fill up the infiltration trench before flowing along the swale. Unfiltered runoff into the next section of the swale has to pass all the way down to the next check dam before it is infiltrated at the next apron.

Infiltration performance is addressed by the infiltration trench routine discussed in Chapter 8. The BMP design module of DURMM described Exhibit A presents the calculations of storage and infiltration as the trench fills up and discharges from storm to storm. Details of the typical bioswale incorporating an infiltration trench are displayed in Appendix B. Design standards, construction specifications and details of the infiltration trench design are set forth the Green Technology Standards, Specifications and Details, Appendix B.

In this manner, a bioswale incorporating a series of cascading dams and infiltration trenches can accomplish remarkable results in terms of filtration, infiltration, streambank protection, and peak flow controls, all in one BMP. As a linear feature roughly 18 feet wide, the multipurpose bioswale is not that much wider than the landscaped islands already required for parking lots. Landscaped with trees along the side slopes, bioswales can be easily integrated into the overall site design without excessive loss of usable ground. It is important though, that enough light remains to permit a dense grass cover. At maturity, the trees will provide enough shade to reduce elevated temperatures in swale runoff. Combined with the savings of land as well as expense from not having to construct a structural BMP, this integrated approach is actually more cost effective, not to mention far healthier to the environment.

CHAPTER 11 BIORETENTION BMPs

11.1 BIORETENTION BMP BACKGROUND

The preceding vegetative filtering BMPs are the first line of defense in the nonstructural BMP approach. If thoughtfully incorporated into the site design, biofiltration BMPs can reduce pollutants to acceptable levels by themselves. However, in more intense urban development, where space may be at a premium and/or the runoff EMCs are elevated, additional nonstructural BMPs are necessary.

Bioretention BMPs are a very effective recent development in BMP design. These "living filters" comprise an organic sandy loam at least two feet deep, covered with a layer of mulch and vegetation. Generally located off-line in small depressions designed to intercept the quality storm volume, filtering occurs as runoff percolates through the mulch and soil matrix into an underdrain. Where infiltration rates are high, exfiltration from the facility can replace the need for underdrains. Bioretention BMPs are particularly effective for removing metals and TSS, and phosphorus to a lesser extent. Nitrogen reduction is more variable and less effective (Coffman and Winogradoff, 1998).

Since much of the captured runoff is released by evapotranspiration, bioretention facilities provide mass reductions even greater than that removed by their reduction in EMCs. Underdrain effluent from bioretention facilities can be up to 12º C cooler than the temperature of incoming runoff (Davis and others, 2000b), providing excellent thermal protection to the receiving waters. These aspects of bioretention facilities make them particularly attractive as a nonstructural BMP, particularly for intensively used paved areas.

Depending upon the infiltration rates of the soil matrix, bioretention facilities are typically sized to handle a hydraulic loading rate of 1.5 inches per hour. Since hydraulic loading rates control facility sizing, it is important that the facility be located to capture the most polluted runoff from impervious areas, with as few contributions from pervious areas as is possible. Depending upon the *CN* of the contributing watershed, these factors suggest that bioretention facilities be sized at roughly 5% of the drainage area.

If the landscaping is not flood tolerant, surface ponding should be restricted to less than a foot at most, and surface drainage should occur within hours after the rainfall ends (Coffman and Winogradoff, 1998). This is necessary to ensure that such landscaping will bear the occasional immersion. Therefore, bioretention BMPs should be located off-line so extended detention surcharge from larger storms is not a potential problem. However, if facultative wetland species are used, a greater range of flooding regimes is acceptable. Not only can such plants tolerate greater depths for longer times, they can also function in hydrologic conditions that approach constant saturation. For these reasons, DURMM recommends facultative plants for bioretention facilities. As discussed below, denitrifying bioretention facilities require facultative landscaping.

Filtration and adsorption are the two mechanisms of pollutant reduction in bioretention. Adsorption is particularly effective in removing the soluble metals that are not susceptible to filtration. As such, the proper composition of the soil matrix of the facility is very important. Sometimes, additional fines such as sand are needed to ensure adequate percolation rates, but sand has a very low CEC, so soil adsorption potential is low. If there is too much sand, efficiency is reduced (Davis and others 2000b), because either adsorption is too low or infiltration rates are too high. Therefore, the proportion of sand should be only what is needed to provide adequate infiltration, with the balance in an organic soil/compost mix.

11.2 BIORETENTION BMP RESULTS

If the infiltration rates are adequate, laboratory experiments suggest that topsoil is an excellent medium, as organic soils have 15 times the CEC of mineral soils on a weight basis (Brady, 1990). Column tests indicate that dissolved copper, zinc, and lead all have similar adsorption rates within a topsoil matrix. The mulch layer is also particularly important, providing adsorption coefficients three times that of the soil matrix (Davies and others, 2000a). To ensure adequate infiltration rates, the organic soils should be augmented with fines so the final mix is less than 15% clay. For best results, the pH should be close to neutral so zinc adsorption is maximized.

Recently, Davis and others (2000a, 2000b) published two studies of bioretention performance. The first report was a laboratory study of two pilot bioretention boxes comprising a sandy loam topsoil with 0.6% organic matter and a CEC of 29 meq/kg, relatively low values for optimal adsorption. A surface layer of shredded hardwood mulch was interspersed with several junipers to replicate field conditions as accurately as possible. Depths of the two boxes were 2.5 and 3.5 feet, with sample ports at intermediate depths to extract effluent as it passes through the soil matrix (Davis and others, 2000a).

A synthetic urban runoff "cocktail" was used to ensure uniform input concentrations. Applied at 4.1 cm/hr for 6 hours, this application rate was designed to represent runoff from a rainfall event of 1.5 cm over a drainage area with a runoff coefficient of 0.8 and 20 times the surface area of the facility. While this application volume is only 30% of the quality storm depth of 5.0 cm recommended in Chapter 4, it is applied over 6 hours instead of the 24-hour distribution of the quality event. If the first 10 hours of the latter event are essentially dry, this represents an average rainfall rate of 0.36 cm/hr over 14 hours, similar to the 0.25 cm/hr rate used in the experiment.

Nonetheless, surface ponding up to 7 inches deep was observed once the facilities had become saturated. While the shallower box percolated at 1 to 2 cm/hr, infiltration rates in the deeper box were as low as 0.3 to 0.4 cm/hr, and ponding remained for roughly two days after application. However, drainage from the boxes was restricted to the small diameter observation port, instead of being allowed to freely drain downward into an underlying soil or underdrain, as would be the case in the field, so these slow rates may be an artifact of the experimental design. As both boxes were comprised of identical soils, this also highlights the inherent variability of soils, suggesting that the proper composition of, and low compaction, of a thoroughly mixed soil matrix is essential.

Results from the effluent sampling in the pilot study are displayed in Table 11-1. Output concentrations are calculated as the average reduction percentage times input concentration. Except in the case of phosphorus, ammonia, and nitrate, reductions were high in all sampling ports. Applied as soluble P, the phosphorus reductions were not nearly as high in the upper ports as that seen in the output effluent. Phosphorus reductions were primarily due to adsorption. This is the primary relationship involved in determining the required depth of the facility, as depths of less than 2 feet provided substantially less reduction in phosphorus levels.

In this analysis, organic N is allocated as the residual of TKN less ammonia. Organic N reductions varied from 84% to –64%, suggesting minimal reductions on average. Whether dissolved or particulate, the widely varying reductions of organic N concentrations do not seem to follow the trends discussed in Chapter 3, where concentrations of organic N are relatively low in base flow, even where they are high in surface runoff. As a substantial component of urban runoff, organic N responses to bioretention are a need for future research. Nitrate reduction was actually negative in the upper ports, with minor efficiency ratios overall. Negligible reductions in ammonia levels were also observed in the upper ports, but since ammonia is adsorbed onto soils, reductions had increased substantially by the bottom. These results suggest that organic N and ammonia captured in the upper layers are transformed into nitrate between tests (Davies and others, 2000a), a conclusion others have reached for the filtration results discussed above.

Analysis of the filter matrix indicated that the mulch retained very high concentrations of metals compared to the soils. However, since mulch is a much smaller proportion than soils by weight, it retained only 20% of the applied metals, with the balance remaining in the soils. This suggests that regular replacement of the mulch layer would improve performance and useful life. Accumulation rates in the soils indicated a useful life of nearly 60 years before the soils matrix becomes saturated, or longer if the mulch is replaced regularly.

Table 11-1: Input and Output EMCs and Efficiency Ratios in Pilot Bioretention Tests

PARAMETER	NO3	NH4	TKN	ON	SP	Zn	Pb	Cu
AVERAGE INPUT CONCENTRATION (mg/l)								
SMALL		1.20	3.50	2.30	0.44	0.600	0.061	0.140
LARGE	0.34	2.40	2.80	0.40	0.52	0.590	0.054	0.064
AVERAGE OUTPUT CONCENTRATION (mg/l)								
SMALL		0.48	0.88	0.40	0.13	<0.025	<0.002	<0.002
LARGE	0.26	0.50	0.90	0.39	0.10	<0.025	<0.002	0.005
EMC REDUCTION								
SMALL		60%	75%	83%	71%	>97%	>98%	98%
LARGE	24%	79%	68%	2%	81%	>98%	>98%	92%

Following the pilot study, the same research team studied field performance of two bioretention facilities in Maryland. The Greenbelt facility constructed in 1992 incorporated well-established grass and shrubs on a mostly soil matrix. The Landover facility constructed in 1998 had less landscaping on a filter media of 50% sand, 20-30% topsoil, and 20-30% leaf mulch. Both facilities were tested with the same synthetic runoff at the same application rate as the pilot tests. Antecedent moisture conditions were favorable for infiltration (Davies and others, 2000b). Results are displayed in table 11-2.

Note that the Greenbelt facility performed as well as the pilot studies, all of which used soils as the media. However, the Landover facility had poorer performance in terms of metals reduction. The authors note that even though the Landover facility had a good mulch layer, which should have effectively removed metals, observed reductions were less than optimal. The possibility of preferential flow through the porous media was discounted since nutrient reductions were similar in each test. However, they also noted that the Greenbelt facility had a higher soil to fines ratio, increasing the potential for adsorption. Since all of the better performing facilities had a topsoil media, this suggests that the mixture used in the Landover facility could be improved. This is an important issue for future research.

Table 11-2: Input and Output EMCs and Efficiency Ratios in Field Bioretention Tests

PARAMETER	NO3	NH4	TKN	ON	SP	Zn	Pb	Cu
INPUT CONCENTRATION (mg/l)								
GREENBELT	0.33	2.60	3.5	0.90	0.52	0.53	0.042	0.066
LANDOVER	1.30		6.9		0.83	1.10	0.054	0.120
OUTPUT CONCENTRATION (mg/l)								
GREENBELT	0.28	0.22	1.65	1.47	0.19	<0.025	<0.002	<0.002
LANDOVER	1.10		2.3		0.11	0.39	0.016	0.069
EMC REDUCTION								
GREENBELT	16%	92%	52%	-64%	65%	>95%	>95%	97%
LANDOVER	15%		67%		87%	64%	70%	43%

Hsieh and Davis (2202) recently reported results for a subsequent study of bioretention in columns. In this study, columns were filled with a variety of different media and the resulting reduction efficiencies measured. Removal of oil and grease was excellent (greater than 99 percent) as was the removal of lead, which was adsorbed onto TSS, that was also removed at high rates. Nitrate removal was quite low (1 percent to 43 percent). Sand performed very poorly for nitrate, while mulch performed the best.

Phosphorus removal was variable, ranging from 5 percent to over 80 percent. It was noted that mulch dominated columns had poor removal rates and slow infiltration rates, while sand had 80 percent removal and high infiltration rates. For this reason, mulch is not considered a good medium for phosphorus removal. Since it also decomposes rapidly, mulch is not recommended as part of the bioretention media, unless nitrate removal is identified as a problem, and mulch can be reliably added on a regular basis to replace that lost by decomposition.

This relationship results in a counterintuitive trend where the better the infiltration rate was, the better the removal rates were. Even though sand removed phosphorus at high rates in this study, it seems unlikely that it would continue to perform at such high rates once its CEC sites become saturated. Dissolved orthophosphate would then pass through unimpeded. For this reason, proportions of sand over 90 percent are not recommended.

11.3 BIORETENTION BMP KINETICS

The preceding discussion demonstrates the excellent potential of bioretention facilities for removing metals, soluble P, and ammonia from urban runoff. Combining the better results from Tables 11-1 and 11-2, Table 11-3 sets forth the irreducible concentrations and maximum reduction percentage possible from bioretention facilities. Since DURMM does not account for N transformations, the maximum nitrate efficiency ratio is 20%, and organic N reductions are estimated at 25%. Although there were no measurements of TSS and particulate P reductions, the metals reduction percentage suggest similar reductions for TSS. Since some of the particulate P will undergo transformations to soluble forms, its net reductions would be less than TSS but more than soluble P. It is thus estimated at 85% in this analysis.

At a given depth, there does not seem to be any design factors that provide incremental changes in efficiency ratios. Instead, two thresholds should be met in all designs: a minimum soil media depth of 2.5 feet (76 cm) for optimal phosphorus reduction; and a maximum loading rate of 1.5 inches/hour over a 12 hour period for a one inch event. Additional research may provide data to ascertain if flushing effects occur at higher rates. Even though one pilot facility had ponding problems with a 6-hour period, this is likely to be due to drainage from the facility being restricted to the small diameter observation port. Since there were no problems with ponding on the field studies, the design loading rate and 12 hour application period seems reasonable. As discussed above, the proper composition of the filter media is essential.

As Table 11-3 indicates, bioretention facilities can provide excellent reductions in metals, phosphorus and TSS. However, since nitrate can comprise much of the total N in urban runoff, the typical aerobic bioretention facility could still release unacceptably high levels of nitrate where the receiving waters are nitrate limited.

Table 11-3: Maximum Efficiency Ratios and Minimum EMCs for Bioretention Facilities

POLLUTANT	TSS	PP	SP	ON	NH4	NO3	CU	ZN
MAXIMUM REDUCTION	97%	95%	85%	85%	90%	24%	99%	99%
MINIMUM EMC (mg/l)	3	0.02	0.10	0.35	0.20	0.25	0.002	0.002

The BMP design module of DURMM described in Appendix A provides estimates on bioretention facility performance as a function of loading rate, pollutant levels, length, width, and side area. Infiltration performance is addressed by the infiltration trench routine discussed in Chapter 8. The BMP design module of DURMM described Exhibit A presents the calculations of storage and infiltration as the facility fills up and discharges from storm to storm. Details of the typical infiltration trench are displayed in Appendix B. Design standards, construction specifications and details of the bioretention facility design are set forth the Green Technology Standards, Specifications and Details, Appendix B.

11.4 DENITRIFYING BIORETENTION

To address the issue of nitrate loads, members of Davis team reported results from column tests designed to provide nitrate removal by denitrification (Kim and others, 2000). Experiments were designed to screen out the best carbon/electron source from among the candidates alfalfa, newspaper, leaf mulch, sawdust, wood chips, and wheat straw. An inorganic electron donor, sulfur was also evaluated in terms of two different particle sizes, 0.6 to 1.18 mm and 2 to 2.36 mm. Limestone was added to buffer the acid production from sulfur oxidation. 40 cm high by 6.4 cm diameter columns with these media were inoculated with secondary effluent from an activated sludge plant for 2 days. Anoxic synthetic runoff was then introduced at 4 cm/hr. Removals were then measured after 35 to 40 days of retention.

Over this period, newspaper and wood chips were the best organic electron sources, providing nearly complete denitrification without other adverse effects. Leaf mulch was less effective at 60% reduction, while alfalfa released odors, turbidity, and 2-3 mg/l TKN. Wheat straw also had high levels of turbidity and TKN, while wood chips and sawdust had low turbidity. However, sawdust had lower reductions in nitrate levels than the wood chips and newspaper, which approached 100% reduction in nitrate levels.

Of the sulfur experiments, the smallest particle size performed best, with nitrate reductions close to 92%, while the larger particles reduction was 33%. The better results with the smaller particle sizes were attributed to their more than doubled surface to volume ratio. However, there is some concern that some of the nitrate is leaving the system as nitrite. Longer retention times were thought to lower the nitrite levels.

The TKN release from alfalfa and wheat straw was attributed to the relatively low C:N ratio in these materials, which tends to promote conversion of organic N to ammonia (ammonification or mineralization). The authors also noted that sulfate-reducing bacteria could reduce nitrate to ammonia under anaerobic conditions when there is a high carbon to nitrate ratio, which would conserve total N. It was thought that this effect would decline over time.

The hydraulic loading rate and time span of the experiment need to be evaluated in terms of their applicability to urban runoff events. The ratio of event volume to interevent interval controls average runoff loading rates during field conditions. However, the 35 to 40 day time span of the experiment exceeds typical intervals between runoff events by a factor of 10. No data is presented for reductions in nitrate levels at the shorter intervals of interest in BMP design (typically 2-4 days between events). Rate kinetics may be inferred from the sulfur experiments where the extent of denitrification seems proportional to surface area. The decline in reduction with larger size particles suggests that they should be more effective if given a longer retention time; conversely, the smallest particles would be less effective at a shorter time frame. If this were the case, sulfur at any size would seem to be ineffective at the shorter time periods of interest.

Assuming 40% in pore space, 16 cm of runoff would fill the columns in roughly 4 hours. Containing some 500 cc of runoff at a nitrate concentration of 2.0 mg/l, the total load of nitrate N would be 1.0 mg. Normalized by the column area of 0.0032 m^2, this represents a load of 3.1

kg/ha. Given a 16:1 ratio of runoff to rainfall similar to that used in the previous pilot and field experiments, this implies a total rainfall volume of 1 cm, well below the 5.0 cm recommended in Chapter 4. At an annual precipitation capture of 35 cm, total annual denitrification would be roughly 35 times 3.1 kg/ha, or 109 kg/ha/yr.

The likely effects of higher loading rates, N transformations, and shorter retention time can be clarified by analysis of the literature on N processes in the landscape. The required annual denitrification of 109 kg/ha/yr is similar to the average annual denitrification rates in forest soils amended with nitrate (Groffman and Tiedje, 1989). Ammonia in streamflow from agricultural areas is less than 2% of total N (Correll and others, 1994), thus the sulfate reduction pathway does not seem to occur in the field. In agricultural fields where roughly 2% of the organic N pool is mineralized per year, high C:N ratios in cover residues substantially reduce nitrate losses (Baker and Senft, 1992). These and other field studies confirm that high C:N ratios determine the potential for denitrification (Drury and others, 1991, and many others). A similar relationship is likely to exist for the column experiments.

As to the retention time necessary for complete denitrification, denitrification rates in pasture soil cores under conditions optimal for denitrification have been measured as high as 5 kg/ha/day (Colburn, 1993). Normalized over column area, this rate corresponds to 67 µg/hr. However, nitrate concentrations in the soil cores were as high as 24 mg/l, 12 times the 2 mg/l used in the columns. Colburn (1993) proposed the following equation for estimating denitrification rate *D* as a function of nitrate concentration *N*, temperature *T* and soil moisture content *W*:

$$D = Ne^{(0.1W + 0.1T - 8.3)} \tag{37}$$

Assuming that the values of *T* and *W* in the soil cores represent field conditions typical for bioretention facilities, denitrification rates are then directly proportional to nitrate concentrations. This implies that the rates in the columns would be 1/12 of that observed in the field cores, or around 6 µg/hr. Over a 72 hour interevent interval, the amount of nitrate N reduced at this rate would be 0.43 mg, or 43% of the applied N.

Given that nitrate concentrations in urban runoff are typically lower than 2 mg/l, the removal rate in soils under field conditions would be even lower. However, nitrate reduction in soils is localized to microsites within the soil profile (Parkin, 1987), whose activities dominate the denitrification response of soil cores (Christensen and others, 1990). Since the column media is optimized for nitrate reduction throughout, its removal rate per unit area should be considerably higher than soils. Therefore, the potential for reducing nitrate in bioretention facilities using a sawdust media appears to be quite promising, although complete denitrification may not occur between closely spaced large events. As indicated in the pilot and field studies, there does appear to be some nitrate reduction occurring in the topsoil bioretention media.

Kim and others (2000) propose a denitrifying cell below a 2’ bioretention layer, with denitrified effluent discharged from the bottom as it is surcharged from above. However, given a permanently saturated zone at the bottom surface of the bioretention media, capillary suction would induce moisture into the bioretention layer above. This would induce anaerobic conditions to persist for several days after a storm event, or even weeks during winter. Under such circumstances, nitrification of ammonia would be inhibited, reducing potential reduction in total N by the denitrification layer.

To avoid capillary suction, a capillary fringe barrier in the form of several inches of coarse sand would be required between the bioretention and denitrification layers. The required volume of the denitrifying layer should be equal to the design event runoff volume, less soil storage capacity in the bioretention layer. This permits time for denitrification of this volume to occur during the average interevent interval of around three days. Given an average of 20% of the media volume in macropores, a bioretention layer 60 cm deep would provide for 12 cm of storage. At a 40% void ratio, 40 cm of runoff (assuming the same ratio of impervious area to biofiltration area as used above; equal to 16 times the 2.5 cm design event), less 12 cm, would require a denitrification layer 70 cm deep. Design depths would be adjusted in relation to the volume of runoff divided by surface area.

The preceding discussion on denitrification in agricultural soils raises an interesting possibility for the design of bioretention facilities. Since soils denitrify at high rates when saturated, the bioretention layer could incorporate fine sulfur and sawdust as the fines fraction with a topsoil adsorption fraction, overlaid upon a denitrifying layer of saturated sawdust. During saturated conditions, the bioretention soil layer would also denitrify nitrate from incoming runoff and that which has been nitrified from organic N. This would improve interevent efficiency ratios, and perhaps reduce the required volume of the denitrifying layer. Facultative plant species would be required since the root zone would cycle between aerobic and anaerobic conditions, which would stress exclusively upland plant species.

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