**ORANGE COUNTY STORMWATER PROGRAM**

# APPENDIX E1
# BMP EFFECTIVENESS AND APPLICABILITY FOR ORANGE COUNTY

**September 2003**
**(Updated June 2005)**

A cooperative project between the County of Orange, Orange County Flood Control District and the incorporated cities of Orange County.

Prepared by:
RBF Consulting
14725 Alton Parkway
Irvine, Ca 92618

## Executive Summary

### June 2005 Update

In late 2004, the Santa Ana Regional Water Quality Control Board adopted Order No. R8-2004-0021 to regulate short-term groundwater-related discharges in the San Diego Creek / Newport Bay watershed. The Board adopted this watershed-specific permit due to concerns about elevated levels of selenium and nitrogen in groundwaters within the watershed. With respect to selenium, the Order recognized that dischargers of extracted groundwater could not be assured of achieving numeric effluent limitations through reasonable treatment, source control, or pollution prevention measures because such measures are not currently available for short-term groundwater discharges containing selenium. However, the Order also recognized that the dischargers might be able to reduce or even eliminate nitrate and selenium discharges by implementing short-term volume-reduction measures, and therefore the Order required dischargers to investigate potential volume-reduction BMPs as a part of the application process for coverage under the Order. This BMP Effectiveness Report was updated in June 2005 to include volume reducing BMP fact sheets that were developed in early 2005 to help dischargers evaluate and consider such measures as required by the permit. The volume reducing BMP fact sheets are contained in Attachment B to this report.

### Objectives and Scope of Work

The County of Orange, the Orange County Flood Control District and incorporated cities developed the 2000 Drainage Area Master Plan (DAMP) to comply with requirements of their NPDES stormwater permit. Stormwater Best Management Practices (BMPs) developed and implemented by the Permittees, which provide a significant level of pollutant reduction, are described in the DAMP. Although the DAMP provides for the implementation of a successful NPDES Stormwater Program through the BMPs that have been developed, the Permittees recognize that the field of stormwater quality is highly dynamic and that appropriate BMPs within the DAMP must be periodically reviewed and amended in order for the program to remain successful.

This study was commissioned by the Co-Permittees to: 1) review existing information on the broad array of structural stormwater BMPs presently available; and 2) organize and present specific information that will allow the County to properly select, site, design, construct, and maintain the most appropriate and cost-effective BMP for a particular site condition and its corresponding receiving water quality or beneficial use objective. The study reviewed performance experience of both established and innovative BMPs on a local, regional, national, and international basis for their potential applicability to Orange County. The evaluation effort included a review of technical literature, a review of existing control programs and demonstration or research projects, and input from consulting firms and municipalities already involved in control program implementation. Performance review included consideration of pollutant removal effectiveness, cost-effectiveness, long term operational potential, and identification of various design considerations that would facilitate site-specific selection within Orange County.

The work completed as part of this task included:

- **A review of the technical literature** - This review was built on, but not limited to, prior BMP effectiveness evaluations and literature reviews conducted by the consultant team and updated to reflect the current understanding of the design, siting, performance, maintenance requirements and costs of these devices. Previous evaluation reports and databases such as the *California Stormwater BMP Handbooks, Stormwater Best Management Practices in an Ultra-Urban Setting: Selection and Monitoring* (Shoemaker et al., 2000), Evaluation of *Highway Runoff Water Quality* (Young et al., 1996), published Caltrans reports, the ASCE National BMP database and other resources were used in this effort. The goal of the review was to identify candidate BMPs.

- **A review of existing programs and demonstration projects in southern California** - This review assessed the effectiveness of BMPs currently or planned for use in Orange County (the Aliso Creek demonstration projects, Newport Bay watershed basins and Natural Treatment System, among others), other southern California counties and California Department of Transportation (Caltrans) facilities in reducing constituents of concern in urban runoff. The review specifically assessed vector issues associated with the BMPs and the influence of threatened or endangered species and wetland protection regulations on maintenance of the devices. The goal of the review was to determine the actual effectiveness of the candidate BMPs in removing targeted pollutants of concern under a variety of field conditions and as a result prioritize the candidate BMPs.

- **A review of other BMP implementation/assessment projects** - This review considered BMP effectiveness data from other areas, recognizing climatic and other differences. A number of relevant national and international projects that have actually documented the effectiveness of BMPs after they have been implemented were evaluated. The goal of this review was to assess the performance of BMPs that have been implemented and use this data to supplement the other data that is being collected.

**Literature Review of BMPs**

The following candidate BMPs were included in the literature review:

- Extended Detention Basins
- Wet ponds / Constructed Wetlands
- Vegetated Swales
- Vegetated Buffer Strips
- Infiltration Trenches / Soakaways / Filter Drains
- Infiltration Basins
- Bioretention
- Water Quality Inlets (enhanced catch basins)
- Sand and Organic Filters
- Proprietary End-of-Pipe Controls
- Proprietary Drain Inlet Inserts
- Retention / Irrigation

Each of these candidate BMPs was described, a summary of the advantages and limitations were provided, and generally accepted siting and design criteria were presented. Published performance data, maintenance needs, and construction and operation costs also were summarized. Finally, research needs and data gaps were identified for each of the subject technologies.

**Local Experience**

The local Orange County experience with stormwater BMPs was summarized. Various BMPs have been constructed in several areas of Orange County primarily to address the problems related to nutrient loads and bacteria concentrations. These sites include several demonstration projects along Aliso Creek, which was the subject of a Cleanup and Abatement order because of high concentrations of fecal indicator organisms. A number of BMPs have also been constructed in the watershed of Newport Bay, a 303(d) listed waterbody. These BMPs consist primarily of sediment traps located in San Diego

Creek and Upper Newport Bay. In addition, the San Joaquin Marsh was constructed near the head of the bay to reduce nutrient loadings. Finally, a series of "Natural Treatment Systems" are proposed for the watershed to further reduce high bacteria and nutrient concentrations.

These local BMPs differ significantly in design and purpose from those described in the previous chapter. The most significant difference is that facilities intended to reduce nutrient and bacteria are designed primarily to treat dry weather flows. Consequently, these devices can be much smaller than those that would be required under the current permit for treating stormwater flows from the same watershed. Since dry weather flow is present at all of these locations, most of the BMPs are wet ponds or constructed wetlands. Because of the differences in treatment objectives and hydrologic design regimes, the data gained from the BMPs implemented to date in Orange County generally do not help to verify the BMP data from published studies in other areas.

**Performance Summary**

A summary of published performance data was provided to facilitate comparison among the BMPs. There have been numerous studies of pollutant removal in stormwater management facilities, especially in the US. For example, this review identified 38 quantitative chemical monitoring studies of wet ponds alone. Despite the number of monitoring studies, there is an apparent wide range of pollutant removal efficiencies reported. This is likely due to a number of variables including: differences in facility sizing and design features, inadequate or differing QA/QC protocols, different methodologies for calculating removal efficiency, differences in storm runoff influent concentrations, and other factors.

This assessment found that pollutant removal efficiency was an incomplete characterization of treatment performance. Characterization of performance as percent removal may not truly reflect the relative performance of the devices when the influent concentration is relatively low or when the concentration of the BMP effluent is unrelated to influent concentration. For instance, lower percent removal is often reported for low influent concentrations in BMP monitoring studies. In addition, some BMPs, such as sand filters were found to produce a similar effluent quality even though influent concentrations varied dramatically between events. Because of these issues, a performance comparison of BMPs was presented on the basis of outflow concentrations. However, it is acknowledged that facility performance might be best understood by considering both outflow quality and percent removal.

**BMP Recommendations and Selection Procedure**

There are two primary drivers for the implementation of structural BMPs for urban runoff treatment in Orange County: 1) prevent degradation of receiving water quality from new development in what are now relatively undeveloped watersheds; and 2) improve receiving water quality in watersheds impacted by existing development and activities. BMPs recommended for implementation for a specific site should reduce the concentrations and loads of these constituents of concern.

The adverse impacts of urbanization on biological communities, receiving water quality, and habitat are well known and have been summarized by many authors. These changes include higher concentrations of many potentially toxic constituents, degradation of channel bed and bank resulting in habitat loss, and loss of sensitive species. Implementing appropriate structural BMPs will improve discharge quality and reduce peak flow rates, helping to protect receiving waters.

Many of the watersheds in urban areas of Orange County have already been identified as not supporting desired uses, such as contact recreation. Additional development in these watersheds will likely lead to further degradation unless the impacts of this development can be mitigated through the use of structural controls or other measures.

Fecal indicator bacteria and pathogens are clearly one of most important constituents of concern. Of the common public domain BMPs only wet ponds are consistently capable of meeting contact recreation standards for fecal coliform. Nevertheless, substantial reduction in concentration has been observed in many of the other BMPs. In general, no reductions in bacteria concentrations have been reported for vegetated strips and swales.

Metals also often contribute to degradation of receiving waters in Orange County. Many common conventional structural BMPs offer substantial removal of total recoverable metals, with some providing significant reduction of the dissolved phase as well. Substantial reductions in nutrients and dissolved pesticides may be difficult to obtain with current technology.

Selection of the appropriate BMPs for use in Orange County is also subject to local factors such as climate, soil types, and regulatory drivers. The Mediterranean climate of southern California is characterized by distinct wet and dry seasons that can make implementation of vegetated strips and swales more difficult. In addition, many parts of Orange County are underlain by soil types with low permeability, restricting the use of infiltration BMPs. Finally, requirements of regulatory agencies, such as NPDES permits, vector concerns, and endangered species, may impact the applicability of certain BMPs.

An important component of this report is to develop recommendations for the most applicable BMPs for Orange County. The applicability is based on a number of criteria identified through a comprehensive literature review. Factors affecting the inclusion of a BMP on this list include:

- Effectiveness of pollutant reduction
- History of successful implementation elsewhere
- Experience in Southern California with the same or similar BMPs
- Amount and type of maintenance

Based on these criteria, the following BMPs are recommended for consideration for implementation:

- Extended Detention Basins
- Vegetated Swales
- Vegetated Buffer Strips
- Bioretention
- Sand and Organic Filters
- Infiltration Basins
- Infiltration Trenches

These BMPs have been widely implemented and operated successfully in many areas including southern California. One must be particularly careful about the use of infiltration trenches and basins, because of potential issues related to groundwater contamination. In addition, clogging of these devices make be difficult to mitigate unless the underlying soils and groundwater levels are optimal for their use.

In the areas where dry weather flows are present for sustaining a perennial pool or the wetland vegetation, the following BMPs may be especially suitable:

- Wet ponds
- Constructed Wetlands

There are several important considerations when considering wet ponds and constructed wetlands. One is the potential benefit of these devices comes from treating dry weather flow from the contributing watershed. These flows often have high bacteria counts, which are substantially reduced in wet ponds and wetlands. In addition, open water is often viewed as an amenity by local property owners. On the other hand, vector control may require substantial annual maintenance, making these one of the more expensive BMPs to operate.

Many watersheds in Orange County are already highly developed and contribute to the impairment of the associated receiving water. Although the BMPs listed above should be considered first, pre-existing stormwater conveyance systems and the lack of available space in developed watersheds may greatly reduce the number and type of BMPs that may be appropriate for addressing identified impairments. Consequently, the following small footprint devices may need to be considered, in addition to the conventional controls, for retrofit situations even though their pollutant removal performance may not be comparable to the best conventional public domain BMPs.

- Water Quality Inlets (enhanced catch basins)
- Proprietary End-of-Pipe Controls
- Proprietary Drain Inlet Inserts

The following BMP is not currently recommended for widespread implementation at this time.

- Retention / Irrigation

This technology is relatively new and has been implemented only in the Austin, Texas area. There is no published data on the operation, performance, and maintenance of this technology. In addition, the use of mechanical pumping and irrigation distribution systems may lead to higher maintenance requirements than many other devices.

Finally, a BMP selection matrix was developed to help in the selection of the appropriate BMP for a site. This matrix emphasizes the physical compatibility of the BMP and watershed characteristics. Important factors include the size of the tributary area, soil type, required hydraulic head, aesthetic considerations, footprint size, and others.

## Table of Contents

## List of Figures

## List of Tables

## List of Acronyms

| | |
|---|---|
| ANOVA | Analysis of variance |
| BMP | Best Management Practice |
| Caltrans | California Department of Transportation |
| CDP | Coastal Development Permit |
| CDS | Continuous Deflective Separation |
| DAMP | Drainage Area Master Plan |
| EMC | Event mean concentration |
| MCTT | Multi-Chambered Treatment Train |
| NPDES | National Pollutant Discharge Elimination System |
| O&M | Operation and maintenance |
| TKN | Total Kjeldahl Nitrogen |
| TPH | Total petroleum hydrocarbons |
| TSS | Total suspended solids |
| UK | United Kingdom |
| US | United States |
| USEPA | United States Environmental Protection Agency |
| USFWS | United States Fish and Wildlife Service |
| WQMP | Water Quality Management Plan |
| WQV | Water quality volume |

## E1-1 Introduction

### E1-1.1 Study Background

The County of Orange, the Orange County Flood Control District and incorporated cities developed the 2000 Drainage Area Master Plan (DAMP) to comply with requirements of their NPDES stormwater permit. BMPs developed and implemented by the Permittees, which provide a significant level of pollutant reduction, are described DAMP. Although the DAMP provides for the implementation of a successful NPDES Stormwater Program through the BMPs that have been developed, the Permittees recognize that the field of stormwater quality is highly dynamic and that the BMPs within the DAMP must be revised, deleted or added to in order for the program to remain successful.

This study was commissioned by the Permittees to assess the applicability of existing and new BMPs through:

- A review of the technical literature;
- A review of existing control programs;
- Demonstration or research projects;
- Input from consulting firms and municipalities already involved in control program implementation; and
- Other sources.

The goal of this work is the development of specific information needed by the County to properly select, site, design, construct and maintain the most cost-effective BMP for the particular site and the associated water quality requirements to support contact recreation, aquatic life, and other designated uses. New structural BMPs will be selected from candidate BMPs that have been field-tested and evaluated as to their pollutant removal efficiency and cost effectiveness.

### E1-1.2 Scope of Work

An assessment of the performance and applicability of various structural controls that are being used to treat stormwater runoff regionally, statewide or nationally for their effectiveness and applicability for site-specific implementation within Orange County was performed. The goal was to develop specific information that is needed in order to properly select, site, design, construct and maintain cost effective BMPs based upon site-specific conditions and the requirements of the receiving waters as well as determine the long-term effectiveness of the BMP. The work completed as part of this task included:

- **A review of the technical literature** - This review was built on, but not limited to, prior BMP effectiveness evaluations and literature reviews conducted by the RBF team and updated to reflect the current understanding of the design, siting, performance, maintenance requirements and costs of these devices. Previous evaluation reports and databases such as the *California Stormwater BMP Handbooks, Stormwater Best Management Practices in an Ultra-Urban Setting: Selection and Monitoring* (Shoemaker et al., 2000), Evaluation of *Highway Runoff Water Quality* (Young et al., 1996), published Caltrans reports, the ASCE National BMP database and other resources were used in this effort. The goal of the review is to identify candidate BMPs.

- **A review of existing programs and demonstration projects in southern California** - This review assessed the effectiveness of BMPs currently or planned for use in Orange County (the Aliso Creek demonstration projects, Newport Bay watershed basins and Natural Treatment System, among others), other southern California counties and Caltrans facilities in reducing constituents of concern in urban runoff. The review specifically assessed vector issues associated with the BMPs and the influence of threatened or endangered species and wetland protection regulations on maintenance of the devices. The goal of the review was to determine the actual effectiveness of the candidate BMPs in removing targeted pollutants of concern under a variety of field conditions and as a result prioritize the candidate BMPs.

- **A review of other BMP implementation/assessment projects** - This review considered BMP effectiveness data from other areas, recognizing climatic and other differences. A number of relevant national and international projects that have actually documented the effectiveness of BMPs after they have been implemented were evaluated. The goal of this review was to assess the performance of BMPs that have been implemented and use this data to supplement the other data that is being collected.

This report provides recommendations based upon supportable technical information that will allow the Orange County Permittees to properly select, site, design, construct, maintain and assess the long-term effectiveness of the implemented BMPs.

### E1-1.3 Organization of Report

Chapter 1 provides an overview of the regulatory drivers leading to the requirement for use of structural BMPs for treatment of urban runoff and describes how the existing data will be used to develop recommendations specifically for the Orange County Permittees. Chapter 2 presents the results of the literature review of BMP siting, design, performance and maintenance and includes studies from the United States and Europe. Chapter 3 describes the experiences with BMPs already implemented within the County and how those experiences would modify the conclusions drawn from studies conducted elsewhere. Chapter 4 highlights the issues associated with characterizing the pollutant removal performance of various BMPs and presents the results in a format that allows easy comparison between BMPs for various constituents of concern. Finally, Chapter 5 provides BMP recommendations and a matrix for selecting the most appropriate and cost-effective BMP for a specific site.

## E1-2 Literature Review of BMPs

### E1-2.1 Introduction

This chapter presents the results of an international literature review into the siting, design, performance, maintenance requirements, and cost of various BMPs commonly implemented for treatment of urban runoff. The following criteria were used to identify practices that are included in this review:

- The practice must be a structural control, such as a wet pond, swale, etc.
- The practice may address either water quality or quantity. Where quantity / rate control is the primary function of the practice, the emphasis will be on structures directed at smaller, more frequent events (i.e., less volume than the 2-year, 24-hour event).
- Although on-site practices may be included in the review, the focus will be on practices appropriate for 1.0 acre and larger catchments.

Using these criteria, the following structural controls were identified for review:

- Extended Detention Basins
- Wet ponds / Constructed Wetlands
- Vegetated Swales
- Vegetated Buffer Strips
- Infiltration Trenches / Soakaways / Filter Drains
- Infiltration Basins
- Bioretention
- Water Quality Inlets (enhanced catch basins)
- Sand and Organic Filters
- Proprietary End-of-Pipe Controls
- Proprietary Drain Inlet Inserts
- Retention / Irrigation

Each technology is discussed in a separate chapter, and the review is based initially on the data contained in the U.S. EPA menu of BMPs .
(http://cfpub.epa.gov/npdes/stormwater/menuofbmps/menu.cfm).

## E1-2.2 Extended Detention Basins

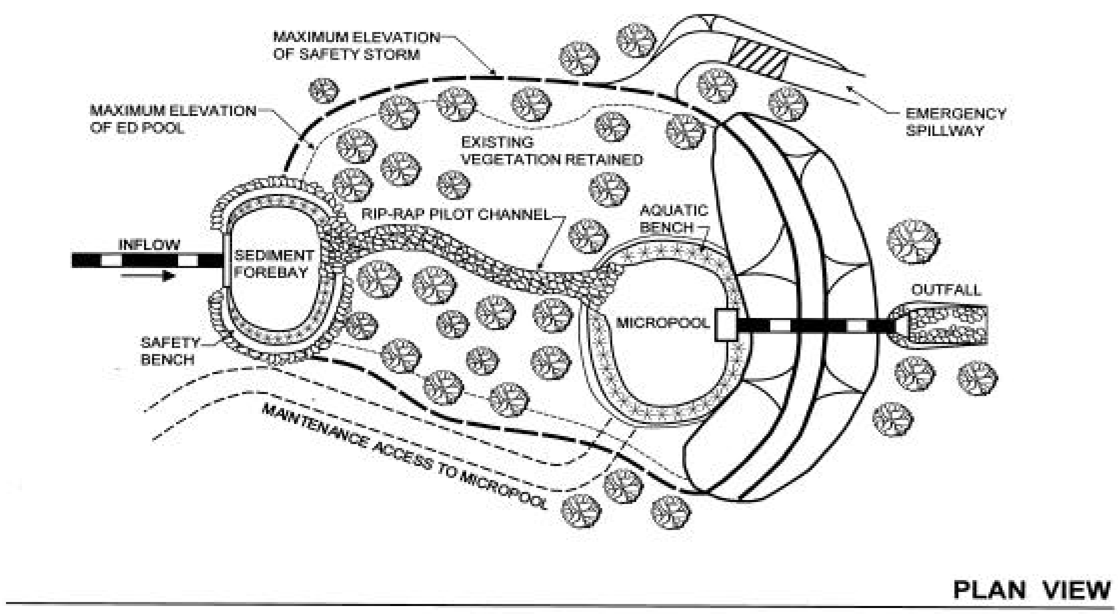
### E1-2.2.1 Description

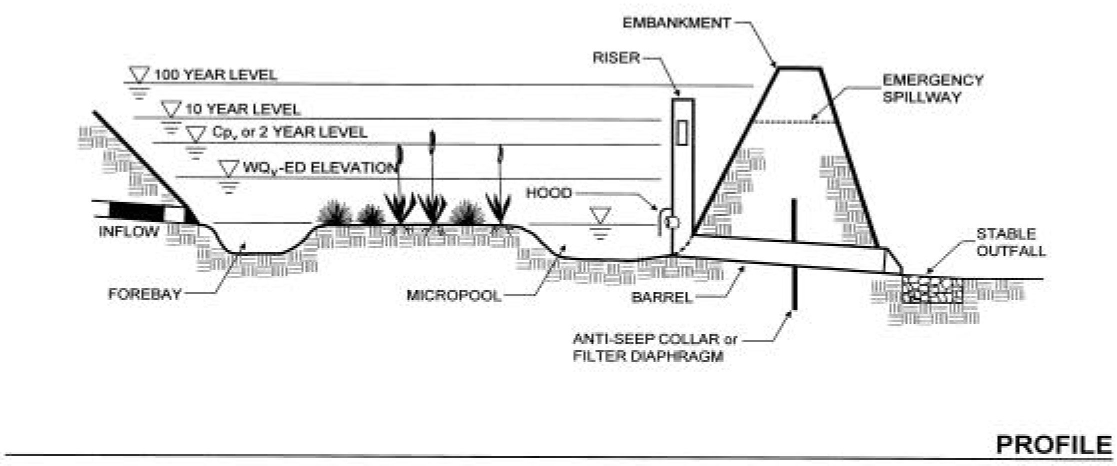
Dry detention basins are normally vegetated depressions, which are dry except during and immediately following storm events. During storm events, surface water runoff is routed through the dry detention basin and the outlet is restricted so that the basin fills with runoff. Dry detention basins are normally used to reduce maximum runoff rates associated with development to their pre-development levels. The control of the maximum runoff rates serves to protect drainage channels below the device from erosion and to reduce downstream flooding. Figure E1-1 shows a detention basin used to treat highway runoff in northern San Diego County.

**Figure E1-1 Dry Detention Basin, I-15/I-78 Intersection, San Diego County**

Dry extended detention ponds are basins whose outlets have been designed to detain the stormwater runoff from a water quality design storm for some minimum time (e.g., 48 hours). Extending the detention times compared to what is required for flood control improves water quality by permitting the settlement of coarse silts and associated pollutants (CIRIA, 2001). These systems may contain a low flow channel and small permanent pool as a landscape feature, and to improve ecological benefits. A schematic of an extended detention basin with a micropool is shown in Figure E1-2.

**Figure E1-2 Schematic of an Extended Detention Basin (MDE, 2000)**



In extended detention basins, the water quality benefits are the removal of sediment and buoyant materials. Furthermore, levels of nutrients, heavy metals, toxic materials, and oxygen-demanding materials associated with the particles may also be significantly reduced. The water quality benefits of a detention dry pond increase by extending the detention time. Substantial removal of total suspended solids (TSS) is possible if stormwater is retained for more than 48 hours. Detention facilities frequently are employed for temporary sediment control during construction, and it may be possible to retain some of these installations permanently (Young et al., 1996).

Dry basins can be considered where the retention of large volumes of water is either not beneficial or inappropriate. It is likely that they are easier to maintain than wet ponds and may be considered less of a health and safety hazard, although the risks must be assessed for wet, dry, and filling conditions. Since there are on average only 40 wet days per year in Orange County, the majority of the time these basins could be available as playgrounds or parks. Dry detention areas can be integrated with, for example, sports fields, thus allowing a dual use where land availability is limited. Consideration should be given to their use in combination with other controls such as grassy swales and vegetated filter strips to achieve better TSS removal.

Although detention facilities designed for flood control have different design requirements than those used for water quality enhancement, it is possible to achieve these two objectives in a single facility. For example, the City of Austin has a dual-purpose facility on Great Northern Blvd., which includes both flood control and water quality elements. A study of the existing flood control facility found that incorporation of water quality components would not affect the primary flood control objectives, so a splitter box was installed to direct the first 0.5 inches of runoff into an extended detention area for slow release to a wetland area.

Although most detention basins are constructed as surface depressions, land costs are driving the use of basins located below grade, underneath parking lots or other structures. These may be constructed on corrugated pipe or of proprietary plastic cells available from a number of manufacturers (e.g., Invisible Structures, Inc.).

### E1-2.2.2 Advantages

- Due to the simplicity of design, extended detention basins are relatively easy and inexpensive to construct and operate.
- Extended detention basins can provide substantial capture of sediment and the toxics fraction associated with particulates.

- Widespread application with sufficient capture volume can provide significant control of channel erosion and enlargement caused by changes to flow frequency relationships resulting from the increase of impervious cover in a watershed.
- Protection of downstream aquatic resources through protection of habitat via peak flow attenuation

E1-2.2.3 Limitations

- Limitation of the diameter of the orifice may not allow use of extended detention on watersheds of less than 5 acres, because it may require very small orifice that would be prone to clogging.
- Requires differential elevation between inlet and outlet.
- Improper design or construction may result in a basin that does not drain adequately and becomes a nuisance to locals, a vector problem and an environmental hazard.
- Not practical for drainage areas over 100 acres
- Freeboard required between spillway and crest of embankment.
- Dry extended detention ponds have only moderate pollutant removal when compared to wet extended detention ponds and some other structural stormwater practices, and they are generally ineffective at removing soluble pollutants. Although wet ponds can increase property values, dry ponds can actually detract from the value of a home.
- Dry extended detention ponds on their own only provide peak flow reduction and do little to reduce overall runoff volume.

E1-2.2.4 Siting Criteria

Dry extended detention ponds are among the most widely used stormwater management practices in the US and are especially appropriate for retrofit situations where their low hydraulic head requirements allow them to be sited within the constraints of the existing storm drain system. Although dry extended detention ponds can be applied rather broadly, designers need to ensure that they are feasible for the site in question. This section provides basic guidelines for siting dry extended detention ponds.

In general, dry extended detention ponds should be used on contributory drainage areas of at least of 5 acres. On smaller sites, it can be challenging to provide channel or water quality control because the orifice diameter at the outlet needed to control relatively small storms becomes very small and thus prone to clogging. In addition, it is generally more cost-effective to control larger drainage areas due to the economies of scale.

The amount of area required for an extended detention basin is highly variable and depends primarily on the design water depth. Young et al. report that these basins normally require between 0.5 and 2 percent of the contributing drainage area for implementation.

Extended detention basins can be used with almost all soils and geology, with minor design adjustments for regions of karst topography or in rapidly percolating soils such as sand. In these areas, extended detention ponds should be designed with an impermeable liner to prevent ground water contamination or sinkhole formation.

Except for the case of hot spot runoff, the only consideration regarding ground water is that the base of the extended detention facility should not intersect the ground water table. A permanently wet bottom may become a mosquito breeding ground and a health and safety risk. Research in Southwest Florida (Santana et al., 1994) demonstrated that intermittently flooded systems, such as dry extended detention ponds, produce more mosquitoes than other pond systems, particularly when the facilities remained wet for more than 3 days following heavy rainfall.

A stormwater retrofit is a stormwater management practice (usually structural) put into place after development has occurred to improve water quality, protect downstream channels from erosion due to increased runoff velocities, reduce flooding, or meet other specific objectives. Dry extended detention ponds are very useful stormwater retrofits, and they have two primary applications as a retrofit design. In many communities in the past, detention basins have been designed for flood control. It is possible to modify these facilities to incorporate features that encourage water quality control and / or channel protection. The only modification normally required would be to reduce the outlet size for smaller, more frequent events. It is also possible to construct new dry ponds in open areas of a watershed to capture existing drainage.

A study in Prince George's County, Maryland, found that stormwater management practices can increase stream temperatures (Galli, 1990). Overall, dry extended detention ponds increased temperature by about 5° F. In cold water streams, dry ponds should be designed to detain stormwater for a relatively short time (i.e., less than 24 hours) to minimize the amount of warming that occurs in the practice.

### E1-2.2.5 Design Guidelines for Extended Detention Basins

Estimating the appropriate dimensions of a BMP facility is largely based on a trial and error process in which the designer tries to fit the required BMP volume so that it works well with the site. Each site has its own unique limiting factors. Some constraints other than the existing topography include, but are not limited to, the location of existing and proposed utilities, depth to bedrock, and location and number of existing trees. The designer can analyze possible basin configurations by varying the surface area and depth and then determining the corresponding available storage (Young et al., 1996).

In some cases optimum capture volume and residence time can be estimated by modeling basin performance using a continuous model based on historical rainfall data. However, many regulatory agencies, including the California Regional Boards have adopted specific sizing criteria that are event based. These requirements reduce the flexibility of designers, but result in more consistent and standardized designs for review.

Many of the design criteria for extended detention basins are not well supported by empirical monitoring. The criteria believed to be the most critical include: time to drain from basin full conditions, basin geometry, and water depth. Although it is common sense that a longer detention time will allow settling of smaller particles, no well designed study has investigated this relationship under field conditions. There have been studies of basins with different drain times, but these facilities also differ in geometry, influent water quality, and design depth. An optimum study would monitor a single facility, changing the outlet orifice periodically to achieve different drain times. This would allow a relationship to be developed between drain time and pollutant removal and indicate how sensitive performance is to this parameter.

Since most of the pollutant removal is associated with particle settling, theory suggests that higher removal can be obtained in a shallower basin, since more particles will reach the bottom before the water is discharged. Unfortunately, a shallower basin can also be more susceptible to pollutant resuspension during subsequent storm events. Consequently, the optimum depth for maximum removal is not well understood.

Likewise, a minimum length to width ratio is often specified in design guidelines to improve performance and reduce short circuiting within the basin. The requirement for a longer flow path is often based on the model of a wastewater clarifier, which operates continuously instead of detention basins that operated more as a batch process. Finally, minimum and maximum depths are often specified in guidelines to improve sedimentation (maximum depth) and reduce resuspension (minimum depth). As with the other factors, no literature that evaluated these parameters was identified.

Despite these uncertainties, US and UK guidelines are generally similar, but requirements in the US are more rigid. Typical criteria are listed in Table E1-1.

A variation of the dry detention pond design is the use of tank storage. In these designs, stormwater runoff is conveyed to large storage tanks or vaults underground. This practice is most often used in the ultra-urban environment, on small sites where no other opportunity is available to provide flood control. Tank storage is provided on small areas because providing underground storage for a large drainage area would generally be cost-prohibitive. Because the drainage area contributing to tank storage is typically small, the required outlet diameter will also be very small creating high clogging potential. Since it is necessary to control small runoff events (such as the runoff from a 1-inch storm) to improve water quality, it is generally infeasible to use tank storage for water quality and generally impractical to use it to protect stream channels.

Detention basins are normally dry, but safety should be considered for both wet and dry conditions. Potential problems arising during the filling of the pond should be assessed. Shallow side slopes and easy access enable people to leave the area as it slowly floods. A safety review should be carried out and, if necessary, a barrier provided. Barrier planting is a preferable alternative to conventional fencing that provides greater value in terms of visual amenity and provision of wildlife habitat. It is used extensively in Scandinavia (CIRIA, 2000).

#### E1-2.2.6 Performance

One objective of stormwater management practices can be to reduce the flood hazard associated with large storm events by reducing the peak flow associated with these storms. Dry extended detention basins can easily be designed for flood control, and this is actually the primary purpose of most detention ponds.

One result of urbanization is the geomorphic changes that occur in response to modified hydrology. Traditionally, dry extended detention basins have provided control of the 2-year storm (i.e., the storm that occurs, on average, once every 2 years) for channel protection. It appears that this control has been relatively ineffective, and recent research suggests that control of a smaller storm might be more appropriate (MacRae, 1996). Slightly modifying the design of flood control basins to reduce the flow of smaller storm events might make them effective tools in reducing downstream erosion.

Dry extended detention basins provide moderate pollutant removal, provided that the design features described in the Siting and Design Considerations section are incorporated. The smaller storm not only contributes to downstream erosion, but also carries the bulk of the pollutant load; therefore, increasing retention time of these smaller events will also facilitate water quality enhancement. Although they can be effective at removing some pollutants through settling, they are less effective at removing soluble pollutants because of the absence of a large permanent pool.

**Table E1-1 Design Guidelines for Extended Detention Basins**

| Design Criteria | Current Guidance/Recommendations |
|---|---|
| Hydraulic Residence Time | • Extended detention facilities should be sized to completely capture the water quality volume and this volume should be increased by a factor of 10% to accommodate reductions in the available storage volume due to deposition of solids in the time between full-scale maintenance activities. |
| Planting | • The landscape design should specify appropriate grass species (to give a dense sward) and plants that will grow well on the site and in the local soils. The choice of species must take into account the fact that the vegetation will be subject to periodic inundation and water flow (CIRIA, 2000). |
| Side Slopes | • The maximum side slope should be designed to meet safety and maintenance requirements (2:1 maximum, less slope preferred). |
| Basin Depths | • Basin depths optimally range from 0.6 to 1.5 m (2 to 5 ft). In the UK, maximum basin depths should not exceed 3 m (10 ft) to avoid damaging the vegetation (CIRIA 2000). |
| Low flows | • A channel should be provided for dry weather flows. |
| Outlet Infrastructure | • The facility's drawdown time should be regulated by a gate valve or orifice plate located downstream of the primary outflow opening.<br>• In general, the outflow structure should have a trash rack or other acceptable means of preventing clogging at the entrance to the outflow pipes. The outflow structure should be sized to allow for complete drawdown of the water quality volume in 48-72 hours with no more than 50% of the water quality volume draining from the facility within the first 24 hours (US recommendations).<br>• UK guidance recommends a total emptying time of the order of 24 hours. The outflow structure should be fitted with a valve (or penstock) so that discharge from the basin can be halted in case of an accidental spill in the watershed. This same valve also can be used to regulate the rate of discharge from the basin.<br>• By using a multi-stage outlet, flows in excess of the treatment volume can be regulated to meet peak flow requirements (CIRIA, 2000).<br>• An overflow should be provided to deal with large storms and to ensure that a minimum freeboard is maintained in the basin (CIRIA, 2000) |
| Inlet Infrastructure | • Energy dissipation and protection should be provided at the inlet to reduce erosion potential in the basin. |
| Pre-treatment | • If excessive sediment loads from runoff upstream have not already been removed before runoff enters the basin, the basin should include a sediment forebay to provide the opportunity for larger particles to settle out.<br>• The forebay volume should be about 10% of the water quality volume and be provided with a fixed vertical sediment depth marker to measure sediment accumulation.<br>• An inlet sediment forebay could consist of a separate basin, or could be formed by building an earthen berm, stone / rock-filled gabion or riprap wall across the upstream portion of the main basin. The plan area should be between 10% and 25% of the total basin area (CIRIA, 2000).<br>• Flow from the inlet basin to the main basins should be arranged so that settled solids are not resuspended and floating litter washed downstream (CIRIA, 2000) |
| Subsoil | • The soils used to finish the side slopes and base of the basin need to be suitably fertile, porous and of sufficient depth to ensure healthy vegetation growth (CIRIA, 2000). |

| Design Criteria | Current Guidance/Recommendations |
|---|---|
| Water Table | • The base of the basin should not intersect any extreme groundwater table, either at the time of the design or at any point in the future.<br>• Where infiltration must be controlled, a liner or membrane should be used. |
| Micropool characteristics | • If a micropool is included in the design, it should be able to store 15 to 25% of the capture volume. The larger end of this range is generally preferred to prevent the micropool from drying out during drought periods. |
| Basin Sizing | • In order to enhance the effectiveness of BMP basins, the dimensions of the basin must be sized appropriately. Merely providing the required storage volume will not ensure maximum constituent removal. By effectively configuring the basin, the designer will create a long flow path, promote the establishment of low velocities, and avoid having stagnant areas of the basin.<br>• To promote settling and to attain an appealing environment, the design of BMP basin should consider the length to width ratio, cross-sectional areas, basin slopes and pond configuration, and aesthetics (Young et al., 1996).<br>• A high aspect ratio improves the performance of detention basins; consequently, the outlets should be placed to maximize the flowpath through the facility. The ratio of flowpath length to width from the inlet to the outlet should be at least 1.5:1 (length / width). |
| Required Footprint | • 0.5 – 2.0% of the tributary area |
| Vehicular Access | • There should be adequate vehicular access to the main basin, inlet and outlet structures, settling pond and the dry weather channel, so that sediment can be removed during maintenance, and to allow grass cutting and vegetation control. |

A few studies are available on the effectiveness of dry extended detention ponds. The California Department of Transportation has conducted one of the more thorough evaluations of extended detention basin performance. Removal rates of four earthen basins are shown in Table E1-2. These basins were all designed to drain from the full condition in 72 hours, had a minimum length to width ratio of 3:1, and captured the 1-year, 24-hour storm (1.0 inches of rainfall). Each of the outlets was designed with multiple orifices to retain the smallest events.

The concentrations shown in Table E1-2 are the mean of the EMCs for the entire monitoring period and include the results from a total of about 55 storm events. The column titled "Significance" indicates the probability that the influent and effluent concentrations are not significantly different, based on an analysis of variance (ANOVA). Since these facilities are located in Los Angeles and San Diego Counties, the removal efficiencies shown are very applicable for Orange County.

One basin in the Caltrans study was concrete lined and often had higher output concentrations than input, which may have resulted from the lack of energy dissipation at the inlet. This illustrates the concern that a detention basin, if poorly designed, can act as a source of pollutants, transporting or re-suspending material, deposited during previous events (Pratt, 2002). Conversely, good design and operation practice may promote higher performance levels than previously observed.

Schueler (1997) also reported on the average effectiveness of a number of detention basins (Table E1-3). Removals are generally better in the Caltrans study except for total nitrogen and bacteria, likely because the detention times were longer than those in most other studies.

**Table E1-2 Pollutant Removal for Extended Detention Basins (Caltrans, 2002)**

| Constituent | Mean EMC Influent mg/L | Mean EMC Effluent mg/L | Removal, % | Significance, P |
|---|---|---|---|---|
| TSS | 137 | 39 | 72 | <0.000 |
| $NO_3$-N | 1.06 | 0.98 | 8 | 0.529 |
| TKN | 2.24 | 1.85 | 17 | 0.206 |
| Total N[a] | 3.30 | 2.83 | 14 | NA |
| Phosphorus | 0.52 | 0.32 | 39 | 0.001 |
| Ortho-phosphate | 0.11 | 0.14 | -22 | 0.332 |
| Particulate P | 0.52 | 0.32 | 39 | <0.000 |
| Total Cu | 0.053 | 0.022 | 58 | <0.000 |
| Total Pb | 0.087 | 0.024 | 72 | <0.000 |
| Total Zn | 0.418 | 0.115 | 73 | <0.000 |
| Dissolved Cu | 0.012 | 0.012 | 0 | 0.899 |
| Dissolved Pb | 0.003 | 0.002 | 29 | 0.078 |
| Dissolved Zn | 0.071 | 0.060 | 16 | 0.279 |
| Particulate Cu | 0.041 | 0.010 | 76 | <0.000 |
| Particulate Pb | 0.084 | 0.022 | 74 | <0.000 |
| Particulate Zn | 0.347 | 0.055 | 84 | <0.000 |
| TPH-Oil [c] | 2.80 | 2.30 | 18 | 0.773 |
| TPH-Gasoline [c] | 0.050[b] | 0.050[b] | NA | 0.974 |
| TPH-Diesel [c] | 1.90 | 1.30 | 32 | 0.321 |
| Fecal Coliform [c] | 900 MPN / 100mL | 2000 MPN / 100mL | -122 | 0.607 |

[a] Sum of $NO_3$-N and TKN-N

[b] Equals value of reporting limit

[c] TPH and Coliform are collected by grab method and may not accurately reflect removal

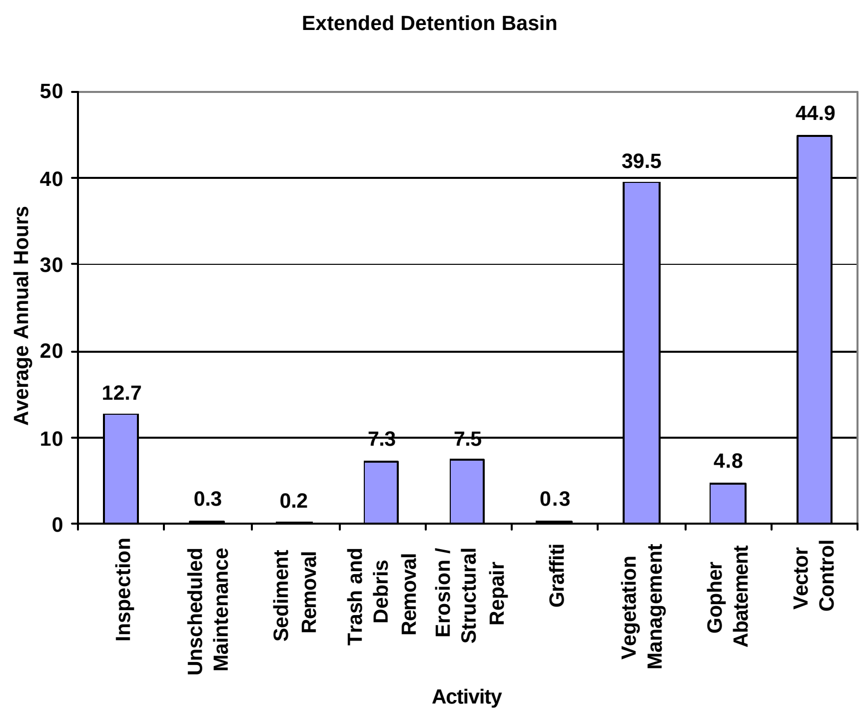
**Table E1-3 Percent Removal for Extended Detention Basins (Schueler, 1997)**

| | TSS | TP | TN | $NO_x$ | HC | Bacteria | Cd | Cu | Pb | Zn |
|---|---|---|---|---|---|---|---|---|---|---|
| Dry ED Pond (6 studies) | 61 | 20 | 31 | -2 | 0 | 78 | 32 | 26 | 54 | 26 |

### E1-2.2.7 Maintenance

Routine maintenance activity is often thought to consist mostly of sediment and trash and debris removal; however, these activities often constitute only a small fraction of the maintenance hours. Figure E1-3 presents the average annual maintenance hours experienced in a recent three-year study by the California Department of Transportation. Individual activities are plotted against the number of person-hours required for completion.

**Figure E1-3 Extended Detention Maintenance Activities**




Of the 72 hours of maintenance performed annually, only a little over 7 hours was spent on sediment and trash removal. The largest recurring activity was vegetation management and routine mowing. The largest absolute number of hours was associated with vector control because of mosquito breeding that occurred in the stilling basins installed as energy dissipaters.

Some activities such as major sediment removal were not performed during the study, but based on the amount of sediment accumulation, this would occur only every 10 or more years.

In addition to incorporating features into the pond design to minimize maintenance, some regular maintenance and inspection practices are needed. Table E1-4 outlines some of these practices.

**Table E1-4 Typical Maintenance Activities For Dry Ponds**

| Activity | Schedule |
|---|---|
| Note erosion of pond banks or bottom.<br>Repair undercut or eroded areas. | Biannually |
| Inspect for damage to the embankment.<br>Monitor for sediment accumulation in the facility and forebay.<br>Examine to ensure that inlet and outlet devices are free of debris and operational. | Quarterly for first year, then annually. |
| Mow side slopes.<br>Manage pesticide and nutrients.<br>Inspect inlet and outlet structures.<br>Remove litter and debris. | Standard maintenance (at least biannually and after large storms). |
| Seed or sod to restore dead or damaged ground cover. | Annually |
| Remove sediment from the forebay. | Post-construction, then 5- to 7-year maintenance. |
| Monitor sediment accumulations, and remove sediment when the pond volume has been reduced by 25 percent. | 25- to 50-year maintenance |

Source: Modified from WMI, 1997

### E1-2.2.8 Cost

#### *E1-2.2.8.1 Construction Costs*

Dry extended detention ponds are the least expensive stormwater management practice, on the basis of cost per unit area treated. The construction costs associated with these facilities range considerably. One recent study evaluated the cost of all pond systems excluding land costs (Brown and Schueler, 1997). Adjusting for inflation, the cost of dry extended detention ponds can be estimated with the equation:

$$C = 12.4V^{0.760}$$

where:

C = Construction, design, and permitting cost, and

V = Volume ($ft^3$).

Using this equation, typical construction costs are:

$ 41,600 for a 1,200 $m^3$ (1 acre-foot) pond

$ 239,000 for a 12,000 $m^3$ (10 acre-foot) pond

$ 1,380,000 for a 120,000 $m^3$ (100 acre-foot) pond

Interestingly, these costs are generally slightly higher than the cost of wet ponds on a cost per total volume basis, which highlights the difficulty of developing reasonably accurate construction estimates. Clearly, dry extended detention ponds are generally less expensive on a given site because they are usually smaller than a wet pond design for the same site.

Table E1-5 presents the construction costs of a number of facilities located across the US compiled by Caltrans (2001) and ranked by normalized cost. It is very apparent from this list that the cost can vary dramatically, so it would not be justified to use the average to estimate the construction cost for a specific site.

An economic concern associated with dry ponds is that they might detract slightly from the value of adjacent properties. One study found that dry ponds can actually detract from the perceived value of homes adjacent to a dry pond by between 3 and 10 percent (Emmerling-Dinovo, 1995).

*E1-2.2.8.2 Maintenance Costs*

For ponds, the annual cost of routine maintenance is typically estimated at about 3 to 5 percent of the construction cost. Alternatively, a community can estimate the cost of the maintenance activities outlined in the maintenance section. Table E1-6 presents the maintenance costs estimated by Caltrans based on their experience with five basins located in southern California. Consistent with the maintenance effort shown in Figure E1-3, the majority of the work is vegetation management that may have little impact on basin functionality and is primarily performed for aesthetic reasons.

### E1-2.2.9 Research Needs

#### *E1-2.2.9.1 Siting Criteria*

Siting criteria for extended detention basins are well established and no additional work in this area is recommended.

#### *E1-2.2.9.2 Design Guidelines*

There are a number of uncertainties in the specific effect of several commonly adopted design guidelines, including detention time, length to width ratio, and optimum depth. Clearly, a longer detention time allows more of the finer particles to settle out; however, there is a maximum practical limit based on the need to prevent mosquito breeding (72 hours) and to have available storage for subsequent storms. A related issue is a hydrological analysis to assess the frequency of complete filling and how climate change might affect that frequency. Basin geometry and depth are likely of secondary importance in regards to performance (± 10 percent?); consequently, assessment of the effect of these design variables is of less importance. A variety of outlets are recommended for detention basins and additional information to identify the preferred configuration which minimizes maintenance would be useful. Finally, some guidance on integration of these facilities with adjacent areas would provide useful information to project designers.

**Table E1-5 Detention Basin Construction Costs ($)**

| Entity | Drainage Area Acres | Water Quality Volume, $ft^3$ | Adjusted Total Cost, $ | Adjusted Total Cost per Acre Treated, $ |
|---|---|---|---|---|
| Maryland & Virginia CWP | 35.00 | 16,155 | 12,558 | 359 |
| Maryland & Virginia CWP | 77.00 | 36,435 | 34,882 | 448 |
| Maryland & Virginia CWP | 380.80 | 1,434,431 | 382,556 | 1,005 |
| Maryland & Virginia CWP | 201.00 | 483,516 | 314,033 | 1,562 |
| MD SHA | 145.31 | 253,200 | 267,680 | 1,842 |
| Maryland & Virginia CWP | 3.10 | 5,663 | 6,813 | 2,198 |
| Maryland & Virginia CWP | 2.30 | 7,841 | 6,813 | 2,962 |
| Maryland & Virginia CWP | 25.00 | 222,447 | 78,536 | 3,141 |
| Maryland & Virginia CWP | 59.10 | 655,523 | 210,793 | 3,567 |
| Maryland & Virginia CWP | 4.50 | 9,200 | 16,464 | 3,659 |
| Maryland & Virginia CWP | 10.90 | 36,590 | 40,752 | 3,739 |
| Maryland & Virginia CWP | 44.30 | 187,308 | 165,652 | 3,739 |
| Maryland & Virginia CWP | 4.30 | 11,258 | 17,507 | 4,071 |
| Maryland & Virginia CWP | 19.50 | 100,188 | 83,875 | 4,301 |
| Maryland & Virginia CWP | 3.10 | 10,600 | 13,626 | 4,396 |
| Maryland & Virginia CWP | 229.90 | 3,571,920 | 1,041,912 | 4,532 |

| Entity | Drainage Area Acres | Water Quality Volume, ft³ | Adjusted Total Cost, $ | Adjusted Total Cost per Acre Treated, $ |
|---|---|---|---|---|
| Maryland & Virginia CWP | 3.20 | 28,314 | 19,091 | 5,966 |
| MD SHA | 6.60 | 3,000 | 39,388 | 5,968 |
| Maryland & Virginia CWP | 3.70 | 22,651 | 29,319 | 7,924 |
| City of Austin | 130.00 | - | 1,282,221 | 9,863 |
| MD SHA | 15.41 | 61,500 | 155,706 | 10,104 |
| Maryland & Virginia CWP | 16.50 | 222,156 | 206,571 | 12,519 |
| Caltrans | 6.80 | 13,068 | 127,202 | 18,706 |
| ODOT | 2.90 | - | 71,540 | 24,669 |
| Caltrans | 5.30 | 13,939 | 147,595 | 27,848 |
| Caltrans | 13.40 | 39,640 | 842,925 | 62,905 |
| Caltrans | 4.80 | 8,712 | 339,116 | 70,649 |
| Caltrans | 0.80 | 2,614 | 77,389 | 96,737 |
| *All costs adjusted to LA region. 1999 | | | | |

**Table E1-6 Estimated Average Annual Maintenance Effort**

| Activity | Labor Hours | Equipment & Material, $ | Cost, $ |
|---|---|---|---|
| Inspections | 4 | 7 | 183 |
| Maintenance | 49 | 126 | 2282 |
| Vector Control | 0 | 0 | 0 |
| Administration | 3 | 0 | 132 |
| Materials | - | 535 | 535 |
| **Total** | **56** | **$668** | **$3,132** |

*E1-2.2.9.3 Performance*

There have been numerous studies of pollutant reduction in extended detention basins, so further water quality monitoring is a low priority.

*E1-2.2.9.4 Construction Guidelines and Cost*

The wide range of cost data reported in Table E1-5 indicates that much more information is needed in this area. Do the differences in cost reflect different regulatory requirements concerning inlet and outlet structures, differences in construction techniques, or perhaps site suitability? Identifying the primary elements that drive construction costs upward is a high priority and could help reduce the initial costs associated with these widely used BMPs.

*E1-2.2.9.5 Maintenance Guidelines and Cost*

Maintenance guidelines are one area in which common sense type recommendations are often derived from a common source that may, in fact, have little empirical basis. Many recommendations seem to derive from aesthetic considerations and may have little impact on performance. The only study for which a detailed accounting of maintenance activities was compiled was conducted in a Mediterranean climate by Caltrans and likely is not representative of the frequency or cost of maintenance in other areas. The development of empirical data related to required maintenance activities and their frequency and cost is a high priority.

### E1-2.2.10 References

Brown, W., and T. Schueler, 1997. *The Economics of Stormwater BMPs in the Mid-Atlantic Region.* Prepared for Chesapeake Research Consortium. Edgewater, MD. Center for Watershed Protection. Ellicott City, MD.

California Department of Transportation [Caltrans], 2002. *BMP Retrofit Pilot Program Proposed Final Report*, Report CTSW-RT-01-050 California Department of Transportation, Sacramento, CA (in press).

Caltrans, 2001. *Third Party Best Management Practice Retrofit Pilot Study Cost Review*, Appendix C of *BMP Retrofit Program Proposed Final Report*, California Department of Transportation, Sacramento, CA. (in press).

Construction Industry Research and Information Association [CIRIA], 2000. *Sustainable Urban Drainage Systems- Design Manual for Scotland and Northern Ireland*. CIRIA C521, ISBN 0 86017 521 9, CIRIA C522, ISBN 0 86017 522 7.

Denver Urban Drainage and Flood Control District, 1992. *Urban Storm Drainage Criteria Manual—Volume 3: Best Management Practices*. Denver, CO.

Emmerling-Dinovo, C., 1995. Stormwater Detention Basins and Residential Locational Decisions. *Water Resources Bulletin* 31(3): 515–521

Galli, J., 1990. *Thermal Impacts Associated with Urbanization and Stormwater Management Best Management Practices*. Metropolitan Washington Council of Governments. Prepared for Maryland Department of the Environment, Baltimore, MD.

Hall, M.J., et al., 1993. *Design of Flood Storage Reservoirs*. CIRIA. Book 14. East Kilbride, Scotland. ISBN 0 7506 1057 3.

Jefferies, C. (Ed.), 2001. *SUDS Monitoring Programme.* SNIFFER Report (00)10. Section 3.3 Detention Basins; Section 5.3 Performance Observations – Detention Basins. Edinburgh EH3 6RT.

MacRae, C., 1996. “Experience from Morphological Research on Canadian Streams: Is Control of the Two-Year Frequency Runoff Event the Best Basis for Stream Channel Protection?” in *Effects of Watershed Development and Management on Aquatic Ecosystem,* ed. L. Roesner. American Society of Civil Engineers, Snowbird, UT. pp 144–162.

Santana, F., J. Wood, R. Parsons, and S. Chamberlain, 1994. *Control of Mosquito Breeding in Permitted Stormwater Systems*. Prepared for Southwest Florida Water Management District, Brooksville, FL.

Schueler, T., 1997. “Influence of Ground Water on Performance of Stormwater Ponds in Florida, *Watershed Protection Techniques* 2(4):525–528.

Schueler, T.R., et al., 1991. *A Current Assessment of Urban Best Management practices.* Metropolitan Washington Council of Governments, Washington, DC.

SEPA, EA, (Pubs). (1999, 2001). *Sustainable Urban Drainage An Introduction*. ISBN 1-901322-12-8.

Watershed Management Institute (WMI), 1997. *Operation, Maintenance, and Management of Stormwater Management Systems*. Prepared for U.S. Environmental Protection Agency, Office of Water. Washington, DC.

Young, G.K., et al., 1996. *Evaluation and Management of Highway Runoff Water Quality*, Publication No. FHWA-PD-96-032, U.S. Department of Transportation, Federal Highway Administration, Office of Environment and Planning.

## E1-2.3 Wet Ponds/Constructed Wetlands

### E1-2.3.1 Description

Wet ponds (a.k.a. stormwater ponds, retention ponds, wet extended detention ponds) are constructed basins that have a permanent pool of water throughout the year (or at least throughout the wet season). They differ from constructed wetlands primarily in having a greater average depth and having peripheral vegetation rather than complete cover. Wet ponds and constructed wetlands treat incoming stormwater runoff through settlement and biological uptake. The primary pollutant removal mechanism is via settlement of silts and suspended sediments. Pollutant uptake, particularly of nutrients, also occurs to some degree through biological activity in the pond. However, to date, no long-term mass balance studies of nutrient uptake and release have been undertaken, and nutrient removal through unattended pond planting remains an area of debate. Wet ponds are among the most widely used stormwater practices. While there are several different versions of the wet pond design, the most common modification is the extended detention wet pond, where storage is provided above the permanent pool in order to detain stormwater runoff and promote settling.

Wet basins are designed to support emergent and submerged aquatic vegetation along the shoreline, as well as an active microbial community capable of dissolved constituent consumption. If properly designed and sized, sedimentation processes are also effective for capturing the particulate fraction. For most runoff events, pond inflows replace a portion of the prior water quality volume and are stored and "treated" until displaced by the perennial baseflow or next runoff event.

Wet basin permanent pools are generally designed deep enough to prevent resuspension of sediment particles and are sized corresponding to a design storm runoff volume or a factor of the design treatment volume. A number of criteria are available for determining the permanent pool volume. In the UK, the objective is to provide a residual retention time of 2-3 weeks during the wettest months. As well as allowing time for biological treatment to occur, this will also improve the degree of suspended particulate settlement and ultimate removal (CIRIA, 2000).

Enhanced wet basins can be designed with a forebay where trapped coarse sediment can be easily removed during maintenance activities and with aquatic benches that support a fringe wetland along the shore. Shallow depths along the pond perimeter also mitigate the potential hazard of children running into the pond and drowning. Additional benefits include creation of aquatic, wetland, and terrestrial habitats, and high community acceptance, if designed as an attractive urban amenity. A picture of a wet pond is presented in Figure E1-4; Figure E1-5 shows a schematic representation of a typical wet basin design.

Public safety should be considered during the design of wet basins and potential drowning hazards should be assessed. Shallow side slopes may enable anyone that has fallen into the pool to escape and the designer may consider fencing the site if water depths will exceed one foot. A safety review should be carried out and, if necessary, a barrier provided. Barrier planting is a preferable alternative to conventional fencing since it provides greater value in terms of visual amenity and provision of wildlife habitat.

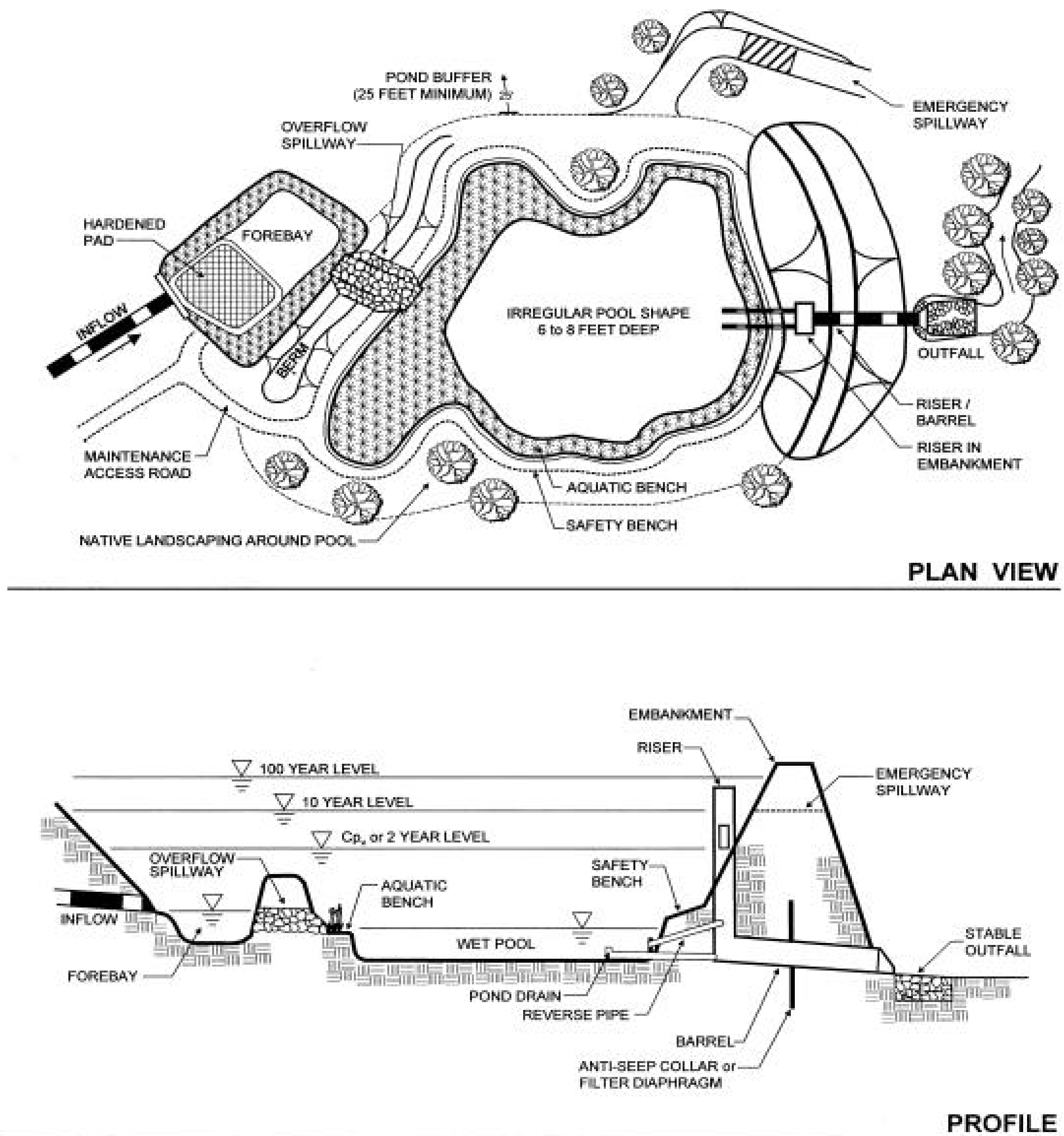
#### E1-2.3.2 Advantages

- If properly designed, constructed and maintained, wet basins can provide substantial aesthetic/recreational value and wildlife and wetlands habitat.
- Ponds are often viewed as a public amenity when integrated in to a park setting.
- Due to the presence of the permanent wet pool, properly designed and maintained wet basins can provide significant water quality improvement across a relatively broad spectrum of constituents including dissolved nutrients.

- Widespread application with sufficient capture volume can provide significant control of channel erosion and enlargement caused by changes to flow frequency relationships resulting from the increase of impervious cover in a watershed.
- The potential for evaporation, transpiration, and infiltration between storms can be important processes for improving treatment performance.

**Figure E1-4 Picture of a Wet Pond, I-5 and La Costa Blvd., San Diego County**

**Figure E1-5 Schematic of a Wet Basin (MDE, 2000)**



### E1-2.3.3 Limitations

Wet ponds are widely applicable stormwater management practices. Although they have limited applicability in highly urbanized settings and in arid climates, they have few other restrictions.

- It may be difficult to use retention ponds within densely urbanized areas because of the land required. However, if space is not available on the site itself, consideration could be given to land immediately downstream.
- In locations where clear water is desired for aesthetic reasons, the value of wet ponds for treatment of nutrient rich runoff is limited, as they may suffer from undesirable algae growth during the summer months.
- Liners may be required to sustain a permanent pool all year round, when wet ponds are sited above permeable soils.
- Where groundwater is vulnerable to pollution, a liner must be used to prevent interaction of any local aquifer with the polluted runoff.
- Careful consideration must be given to the siting and design of a wet pond in areas where groundwater levels are high. The systems may not be suitable in these cases if there is a risk that removing ground cover may facilitate the passage of groundwater to the surface during very wet/flood periods.
- Careful consideration is required to minimize health and safety concerns, where these systems form an integral part of any residential development.
- Wet ponds generally require a contributing watershed of more than 25 acres to provide sufficient runoff to maintain the permanent pool.
- The facility may serve as a habitat for threatened wildlife species, which may require special maintenance and operation procedures.

### E1-2.3.4 Siting Criteria

Wet basins may be constructed on- or off-line and can be sited at feasible locations along established drainage paths with consistent baseflow. In wet climates, basins also perform well without a continuous base flow, providing the pond base is relatively impermeable. Wet basins are often utilized in smaller subwatersheds and are particularly appropriate in areas with residential land uses or other uses where high nutrient loads are considered to be potential problems (e.g., golf courses). Wet basins are typically used in drainage basins of more than 10 acres and less than 1 square mile (Schueler et al., 1992). Emphasis can be placed on siting wet basins in areas where the pond can also function as an aesthetic amenity or in conjunction with other stormwater management functions. Wet basins can be used over a broad range of storm frequencies and sizes, drainage areas and land use types. Wet basins are reported by Urbonas (1992) to require between 1 and 3% of the catchment area for implementation.

Wet basin application is typically appropriate in the following settings:

1. Where there is a need to achieve a reasonable high level of dissolved contaminant removal and / or sediment capture;
2. In small to medium-sized regional tributary areas with available open space and drainage areas greater than about 10 acres;
3. Where baseflow rates or other channel flow sources are relatively consistent year-round;
4. In residential settings where aesthetic and wildlife habitat benefits can be appreciated and maintenance activities are likely to be consistently undertaken.

Although traditionally sited at locations with year round flow, seasonal wet ponds (i.e., ponds that maintain a permanent pool only during the wet season) may prove effective in areas with distinct wet and dry seasons. However, no evaluation of this configuration has not been reported in the literature.

Traditional wet extended detention ponds can be applied across the UK and in most regions of the United States, with the exception of arid climates. In arid regions, it is difficult to justify the supplemental water needed to maintain a permanent pool because of the scarcity of water. Even in semi-arid Austin, Texas, one study found that 3,200 $m^3$ (2.6 acre-feet) per year of supplemental water was needed to maintain a permanent pool of only 350 $m^3$ (0.29 acre-feet) (Saunders and Gilroy, 1997).

Wet ponds may pose a risk to cold water systems because of their potential for stream warming. When water remains in the permanent pool, it is heated by the sun. A study in Prince George's County, Maryland, found that stormwater wet ponds heat stormwater by about 9°F from the inlet to the outlet (Galli, 1990).

Unless they receive heavily polluted runoff, ponds in the US may intersect the ground water table. However, in a number of European countries, such as Germany and the United Kingdom, it is not permitted to infiltrate urban runoff into the ground water table. Some research suggests that pollutant removal is reduced when ground water contributes substantially to the pool volume (Schueler, 1997b); however, this may be the result of groundwater adding substantial dissolved solids to the runoff.

#### E1-2.3.5 Design Guidelines

Specific designs may vary considerably, depending on site constraints or preferences of the designer or community. There are several variations of the wet pond design, including constructed wetlands, and the wet extended detention ponds. Some of these design alternatives are intended to make the practice adaptable to various sites and to account for regional constraints and opportunities. In conventional wet ponds, the open water area should comprise 50 – 75 percent or more of the total surface area of the pond. The permanent pool should be no deeper than 8 feet and should average 4-6 feet deep. The greater depth of this configuration helps limit the extent of the vegetation to the perimeter of the pond. The depth limit prevents anaerobic conditions from developing.

Constructed wetlands generally feature relatively uniformly vegetated areas with depths of one foot or less and open water areas (25-50 percent of the total area) no more than about 4 feet deep, although design configuration options are relatively flexible. Wetland vegetation is generally comprised of diverse, local aquatic plant species. Constructed wetlands can be designed on-line or off-line and generally serve relatively smaller drainage areas than wet ponds.

The wet extended detention pond combines the treatment concepts of the dry extended detention pond and the wet pond. In this design, the water quality volume is split between the permanent pool and detention storage provided above the permanent pool. During storm events, water is detained above the permanent pool and released over 24 to 48 hours. Although not well documented, this design appears to have similar pollutant removal as a traditional wet pond and consumes less space. Wet extended detention ponds should be designed to maintain at least half the treatment volume of the permanent pool. In addition, designers need to carefully select vegetation to be planted in the extended detention zone to ensure that the selected vegetation can withstand both wet and dry periods.

A variation of the wet pond design is the use of tank storage. In these designs, stormwater runoff is conveyed to underground wet vaults. This practice is most often used in the ultra-urban environment, on small sites where no other opportunity is available to provide stormwater treatment. Tank storage is provided on small areas because providing treatment storage for a large drainage area would generally be cost-prohibitive. The use of wet vaults may be problematic because of the potential for mosquito breeding in these structures.

There are some features shown in Table E1-7, however, that should be incorporated into most wet pond designs. These design features can be divided into five basic categories: pretreatment, treatment, conveyance, maintenance reduction, and landscaping.

**Table E1-7 Design Guidelines for Wet Ponds**

| Design Criteria | Current Guidance/Recommendations |
|---|---|
| Inlet Infrastructure | • The inlet should be designed to minimize the velocity of runoff entering the system. Energy dissipation and protection should be provided.<br>• The inlet should be sited to reduce the risk of blockage by vegetation or sediment build-up.<br>• The inlet should be sited as far from the outlet as possible. |
| Pretreatment | • Pretreatment incorporates design features that help to settle out coarse sediment particles. By removing these particles from runoff before they reach the large permanent pool, the maintenance burden of the pond is reduced.<br>• In ponds, pretreatment can be achieved with a sediment forebay. A sediment forebay is a small pool (typically about 10 percent of the volume of the permanent pool). Coarse particles remain trapped in the forebay, and maintenance is performed on this smaller pool, eliminating the need to dredge the entire pond.<br>• Design features should also be incorporated to ease maintenance of both the forebay and the main pool of ponds. Ponds should be designed with a maintenance access to the forebay to ease this relatively routine (5–7 year) maintenance activity. |
| Treatment | • Treatment design features help enhance the ability of a stormwater management practice to remove pollutants. The purpose of most of these features is to increase the amount of time that stormwater remains in the pond.<br>• One technique of increasing the pollutant removal efficiency of a pond is to increase the volume of the permanent pool. There are a variety of sizing criteria for determining the volume of the permanent pool, mostly related to the water quality volume (i.e., the volume of water treated for pollutant removal) or the average storm size in a particular area. In the Orange County area the permanent pool should be at least two times the water quality volume. Designers may consider using a larger volume to meet specific watershed objectives, such as phosphorous removal in a lake system.<br>• Other design features do not increase the volume of a pond, but can increase the amount of time stormwater remains in practice, and eliminate short-circuiting. Ponds should always be designed |

| Design Criteria | Current Guidance/Recommendations |
|---|---|
| | with a length-to-width ratio of at least 1.5:1, although 5:1 has been cited as preferable (Horner et al., 1994) or 3:1 (CIRIA, 2000). In addition, the design could incorporate features to lengthen the flow path through the pond, such as underwater berms / baffles designed to create a longer route through the pond. Windbreaks may also be needed to reduce the influence of the wind on flow patterns.<br>• Combining these two measures helps ensure that the entire pond volume is used to treat stormwater. Another feature that can improve treatment is to use multiple ponds in series as part of a "treatment train" approach to pollutant removal. This redundant treatment can also help slow the rate of flow through the system.<br>• Ponds should generally have a drain to draw down the pond for the more infrequent maintenance activity of dredging the main cell of the pond. Consideration must be given to managing the drainage during these periods of essential maintenance. Areas away from residential sites may need to be set aside for de-watering of the pond sediments prior to disposal off-site. |
| Flood Control | • The additional detention storage provided for flood control should be limited to 2 m above the permanent pool level to avoid damage to the vegetation in the pond (CIRIA, 2000).<br>• An overflow should be provided to deal with large storms and to ensure that a minimum freeboard is maintained in the basin (CIRIA, 2000). |
| Pond Side Slopes | • Maximum pond side slopes should be limited to 1 in 4 to meet safety and maintenance requirements (CIRIA, 2000).<br>• Slope protection may be needed during construction and operation of the basin (CIRIA, 2000).<br>• A vegetated buffer should be preserved around the pond to protect the banks from erosion and provide some pollutant removal before runoff enters the pond by overland flow.<br>• Ponds should also incorporate an aquatic bench (i.e., a shallow shelf with wetland plants) about 3 m wide around the edge of the pond. This feature may provide some pollutant uptake, and it also helps to stabilize the soil at the edge of the pond and enhance habitat and aesthetic value.<br>• Selection of appropriate barrier vegetation can also help discourage toddlers and young children from approaching the pond and can therefore act as an additional safety barrier. |
| Outlet Infrastructure | • The outlet works should provide for a smooth stage-discharge relationship for the discharge of storm flows from the pond. |

| Design Criteria | Current Guidance/Recommendations |
|---|---|
| | This may be achieved, for example, with a V-notch weir transitioning into a sharp crested horizontal weir. Details on the design of different inlet and outlet control structures are provided in Design of Flood Storage Reservoirs (CIRIA, 1993). By using a multi-stage outlet, flows in excess of the treatment volume can be regulated to meet peak flow requirements (CIRIA, 2000).<br>• One potential maintenance concern in wet ponds is clogging of the outlet. Ponds should be designed with a non-clogging outlet such as a reverse-slope pipe, or a weir outlet with a trash rack. A reverse-slope pipe draws from below the permanent pool extending in a reverse angle up to the riser and establishes the water elevation of the permanent pool. Because these outlets draw water from below the top level of the permanent pool, they are less likely to be clogged by floating debris.<br>• The outlet should be sited as far from the inlet as possible. |

*Regional Adaptations*

Semi-Arid Climates

In arid climates, wet ponds are not a feasible option (see Applicability), but they may possibly be used in semi-arid climates if the permanent pool is maintained with a supplemental water source, or if the pool is allowed to vary seasonally. This choice needs to be seriously evaluated, however.

Cold Climates

Cold climates present many challenges to designers of wet ponds. The spring snowmelt may have a high pollutant load and a large volume to be treated. In addition, cold winters may cause freezing of the permanent pool or freezing at inlets and outlets. Finally, high salt concentrations in runoff resulting from road salting, and sediment loads from road sanding, may impact pond vegetation as well as reduce the storage and treatment capacity of the pond.

One option to deal with high pollutant loads and runoff volumes during the spring snowmelt is the use of a seasonally operated pond to capture snowmelt during the winter, and retain the permanent pool during warmer seasons. In this option, proposed by Oberts (1994), the pond has two water quality outlets, both equipped with gate valves. In the summer, the lower outlet is closed. During the fall and throughout the winter, the lower outlet is opened to draw down the permanent pool. As the spring melt begins, the lower outlet is closed to provide detention for the melt event. This method can act as a substitute for using a minimum extended detention storage volume. When wetlands preservation is a downstream objective, seasonal manipulation of pond levels may not be desired. An analysis of the effects on downstream hydrology should be conducted before considering this option. In addition, the manipulation of this system requires some labor and vigilance; a careful maintenance agreement should be confirmed.

Several other modifications may help to improve the performance of ponds in cold climates. Designers should consider planting the pond with salt-tolerant vegetation if the facility receives road runoff. In order to counteract the effects of freezing on inlet and outlet structures, the use of inlet and outlet structures that are resistant to frost, including weirs and larger diameter pipes, may be useful. Designing structures on-line, with a continuous flow of water through the pond, will also help prevent freezing of these structures. Finally, since freezing of the permanent pool can reduce the effectiveness of pond systems, it may be useful to incorporate extended detention into the design to retain usable treatment area above the permanent pool when it is frozen.

### E1-2.3.6 Performance

Structural stormwater management practices can be used to achieve four broad resource protection goals. These include flood control, channel protection, ground water recharge, and pollutant removal. Wet ponds can contribute to all of these objectives, except for ground water recharge.

#### *E1-2.3.6.1 Hydraulic Performance*

One objective of stormwater management practices can be to reduce the flood hazard associated with large storm events by reducing the peak flow associated with these storms. Wet ponds can easily be designed for flood control by providing flood storage above the level of the permanent pool.

When used for channel protection, wet ponds have traditionally provided outlet control for the 2-year storm, while allowing smaller storms to pass without attenuation. It appears that this control has been relatively ineffective, and recent research suggests that control of a smaller storm may be more appropriate (MacRae, 1996). There is usually a trade off between hydrological control and cost, as control of smaller storms will require more surface area for the pond. It may therefore be necessary to rely on flow control upstream of the pond to achieve more ambitious targets. A close look at the geomorphology of the receiving watercourse is necessary to select the appropriate maximum flow rates at the pond outlet. In general, an outlet design that detains the water quality volume for at least 24 hours is preferred in order to provide some protection of the receiving watercourse.

*E1-2.3.6.2 Pollutant Removal*

The observed performance of a wet pond is highly dependent on two factors: the volume of the permanent pool relative to the amount of runoff from the typical event in the area, and the quality of the baseflow that sustains the permanent pool. A recent study (Caltrans, 2002) has documented that if the permanent pool is much larger than the volume of runoff from an average event, then displacement of the permanent pool by the wet weather flow is the primary process. A statistical comparison of the wet pond discharge during dry and wet weather discharge shows that the concentrations are not significantly different. This results in a relatively constant discharge quality during storms that is independent of the influent concentrations. Consequently, for most constituents the performance of the pond is better characterized by the average effluent concentration, rather than the “percent reduction,” which has been the conventional measure of performance. Since the effluent quality is essentially constant, the percent reduction observed is mainly a function of the influent concentrations observed at this particular site.

The dry and wet weather discharge quality is related to the quality of the baseflow that sustains the permanent pool and of the transformations that occur to those constituents during their residence in the basin. One could potentially expect a wide range of effluent concentrations at different locations even if the wet ponds were designed according to the same guidelines, if the quality of the baseflow differed significantly. This may partially explain the wide range of concentration reductions reported in the various reports and Table E1-8 and the following graphs.

There are a number of additional factors contributing to reported variation in performance. The outlet designs of the various ponds likely differ, with some providing detention of the design storm volume and with others having no peak flow attenuation, just displacement of the permanent pool volume. In addition, the permanent pool volume is not consistently related to the design water quality storm. Climatic differences and the average inter-event dry period also likely affect observed performance.

## Table E1-8 Wet Pond Percent Removal Efficiency Data

| Study | TSS | TP | TN | $NO_3$ | Metals | Bacteria | Practice Type |
|---|---|---|---|---|---|---|---|
| City of Austin, TX 1991. Woodhollow, TX | 54 | 46 | 39 | 45 | 69-76 | 46 | wet pond |
| Driscoll 1983. Westleigh, MD | 81 | 54 | 37 | - | 26–82 | - | wet pond |
| Dorman et al., 1989. West Pond, MN | 65 | 25 | - | 61 | 44–66 | - | wet pond |
| Driscoll, 1983. Waverly Hills, MI | 91 | 79 | 62 | 66 | 57–95 | - | wet pond |
| Driscoll, 1983. Unqua, NY | 60 | 45 | - | - | 80 | 86 | wet pond |
| Cullum, 1985. Timber Creek, FL | 64 | 60 | 15 | 80 | - | - | wet pond |
| City of Austin, TX 1996. St. Elmo, TX. | 92 | 80 | 19 | -17 | 2–58 | 89-91 | wet pond |
| Horner, Guedry, and Kortenhoff, 1990. SR 204, WA | 99 | 91 | - | - | 88–90 | - | wet pond |
| Horner, Guedry, and Kortenhoff, 1990. Seattle, WA | 86.7 | 78.4 | - | - | 65–67 | - | wet pond |
| Kantrowitz and Woodham, 1995. Saint Joe's Creek, FL | 45 | 45 | - | 36 | 38–82 | - | wet pond |
| Wu, 1989. Runaway Bay, NC | 62 | 36 | - | - | 32–52 | - | wet pond |
| Driscoll 1983. Pitt-AA, MI | 32 | 18 | - | 7 | 13–62 | - | wet pond |
| Bannerman and Dodds, 1992. Monroe Street, WI | 90 | 65 | - | - | 65–75 | 70 | wet pond |
| Horner, Guedry, and Kortenhoff, 1990. Mercer, WA | 75 | 67 | - | - | 23–51 | - | wet pond |
| Oberts, Wotzka, and Hartsoe 1989. McKnight, MN | 85 | 48 | 30 | 24 | 67 | - | wet pond |
| Yousef, Wanielista, and Harper 1986. Maitland, FL | - | - | - | 87 | 77–96 | - | wet pond |
| Wu, 1989. Lakeside Pond, NC | 93 | 45 | - | - | 80–87 | - | wet pond |
| Oberts, Wotzka, and Hartsoe, 1989. Lake Ridge, MN | 90 | 61 | 41 | 10 | 73 | - | wet pond |
| Driscoll, 1983. Lake Ellyn, IL | 84 | 34 | - | - | 71-78 | - | wet pond |
| Dorman et al., 1989. I-4, FL | 54 | 69 | - | 97 | 47–74 | - | wet pond |
| Martin, 1988. Highway Site, FL | 83 | 37 | 30 | 28 | 50–77 | - | wet pond |
| Driscoll, 1983. Grace Street, MI | 32 | 12 | 6 | -1 | 26 | - | wet pond |
| Occoquan Watershed Monitoring Laboratory, 1983. Farm Pond, VA | 85 | 86 | 34 | - | - | - | wet pond |
| Occoquan Watershed Monitoring Laboratory, 1983. Burke, VA | -33.3 | 39 | 32 | - | 38–84 | - | wet pond |
| Dorman et al., 1989. Buckland, CT | 61 | 45 | - | 22 | -25 to -51 | - | wet pond |
| Holler, 1989. Boynton Beach Mall, FL | 91 | 76 | - | 87 | - | - | wet pond |
| Urbonas, Carlson, and Vang 1994. Shop Creek, CO | 78 | 49 | -12 | -85 | 51–57 | - | wet pond |
| Oberts and Wotzka, 1988. McCarrons, MN | 91 | 78 | 85 | - | 90 | - | wet pond |
| Gain, 1996. FL | 54 | 30 | 16 | 24 | 42–73 | - | wet pond |
| Ontario Ministry of the Environment, 1991. Uplands, Ontario | 82 | 69 | - | - | - | 97 | wet extended detention pond |

| Study | TSS | TP | TN | $NO_3$ | Metals | Bacteria | Practice Type |
|---|---|---|---|---|---|---|---|
| Borden et al., 1996. Piedmont, NC | 19.6 | 36.5 | 35.1 | 65.9 | -4 - 97 | -6 | wet extended detention pond |
| Holler, 1990. Lake Tohopekaliga District, FL | - | 85 | - | - | - | - | wet extended detention pond |
| Ontario Ministry of the Environment 1991. Kennedy-Burnett, Ontario | 98 | 79 | 54 | - | 21–39 | 99 | wet extended detention pond |
| Ontario Ministry of the Environment 1991. East Barrhaven, Ontario | 52 | 47 | - | - | - | 56 | wet extended detention pond |
| Borden et al., 1996. Davis, NC | 60.4 | 46.2 | 16 | 18.2 | 15–51 | 48 | wet extended detention pond |
| Hopwood - Pond 2 (Folkes-Skinner 2002) | 18 | | | 30 | Cd, Ni: 0; Cr: 15; Zn: 65; Pb: 72; Cu: 32 | | Wet pond |
| Summary of UK data - Urban Runoff (JB Ellis *et al.*, 2001) | 55 | | | 29 | | | Wet pond |
| Summary of UK data - Highway Runoff (JB Ellis *et al.*, 2001) | | | | | Zn: 38; Pb: 52 | | Wet pond |
| Newbury - Pond B2 (Shutes et al., 2001) | -6 | | | | Cd: 8, Ni: 52; Cr: 15; Zn: 29; Pb: 0; Cu: -75 | | Wet pond |
| Claylands Pond (Jefferies. C., *et al*., 2001.) | | | | | Ni: 61; Cr:80 ; Zn:42 ; Pb:75 ; Cu: 77 | | |

Median removal rates from the studies listed are:

*Total Suspended Solids: 78%*
*Total Phosphorus: 47%*
*Total Nitrogen: 32%*
*Nitrate Nitrogen: 32%*
*Metals: 60%*
*Bacteria: 70%*

Figure E1-6 through Figure E1-9 graphically illustrate wet pond performance for selected constituents.

**Figure E1-6 TSS Removal in Wet Ponds**

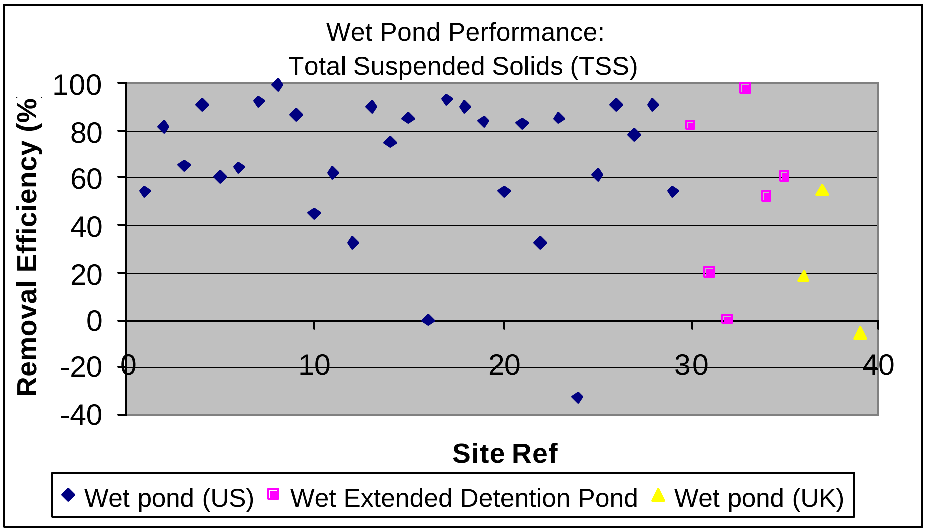



**Figure E1-7 Phosphorus Removal in Wet Ponds**

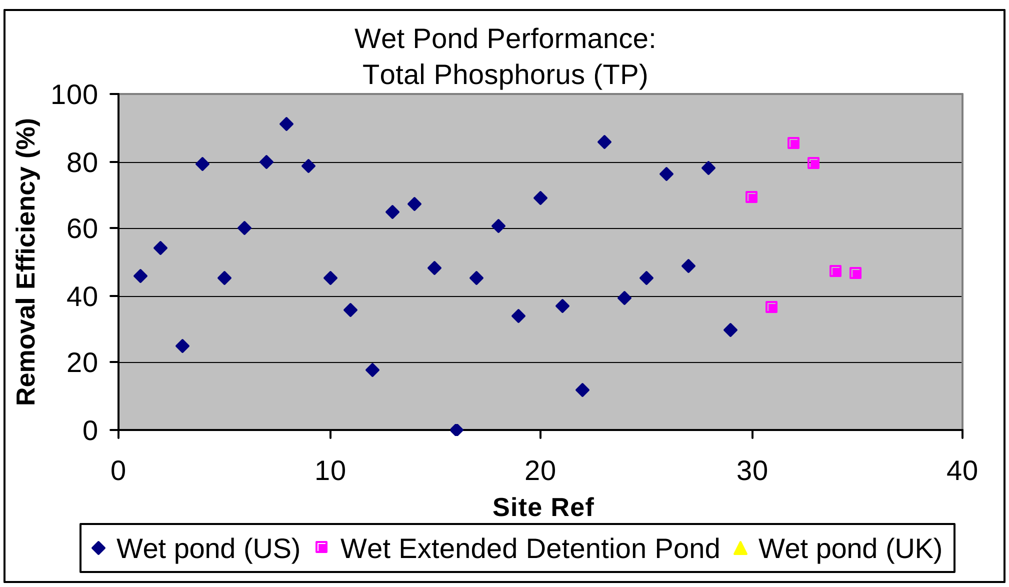


**Figure E1-8 Total Nitrogen Removal in Wet Ponds**

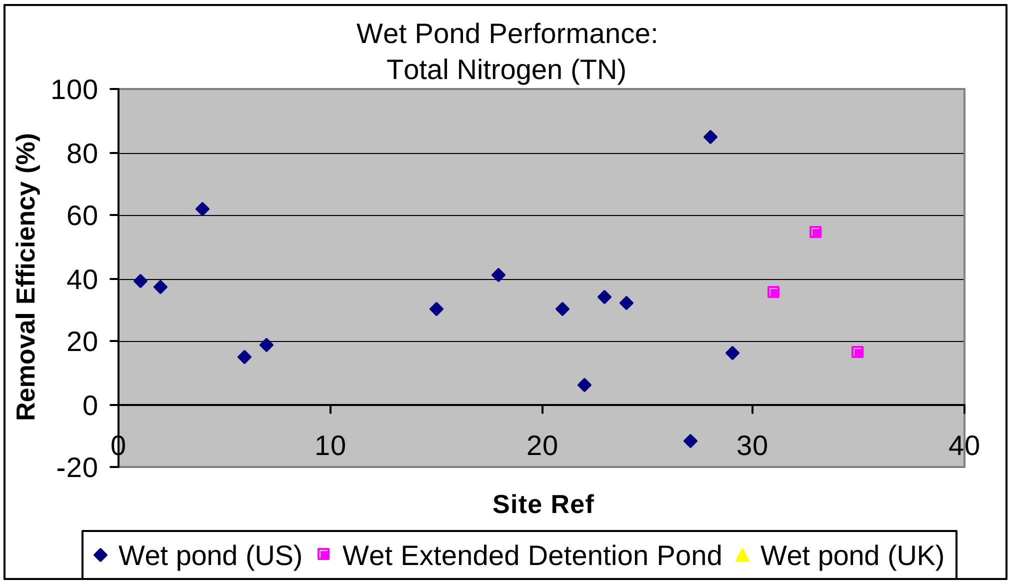
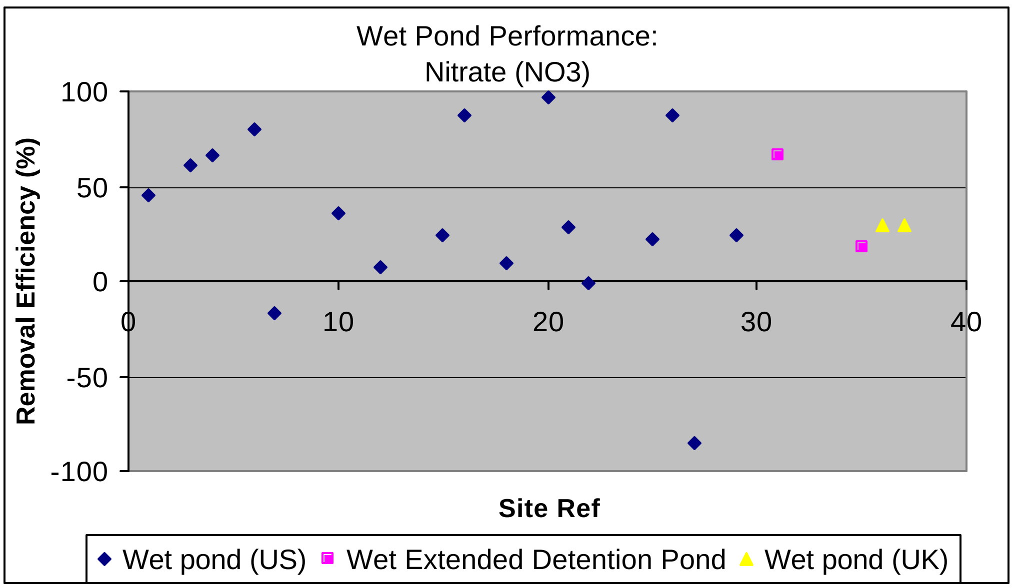



**Figure E1-9 Nitrate Removal in Wet Ponds**



Most of the data indicate substantial and consistent removal of TSS; however, the data for nutrients shows much higher variability. Reasons for this variability were described by O'Shea et al. (2002) and are reproduced below.

Several factors will affect the extent of ammonification, nitrification, and denitrification. Some of the more important include: the form, concentration and timing of nitrogen loads; the macro- and microbiotic communities; the availability of a carbon supply sufficient to fuel denitrification; and the chemical and physical conditions (e.g., temperature, pH, and dissolved oxygen) within a system. Export, rather than retention, of any single species may occur in the absence of conditions or reactants necessary to maintain reaction rates. For example, wetlands receiving influent high in $NH_4^+$ or ON, rather than $NO_3^-$ have been observed to export TN. Similarly, an inadequate or unavailable carbon source in newly created or subsurface flow wetlands may limit denitrification, leading to $NO_3^-$ export.

In contrast to the dissolved inorganic phosphorus (DIP) species, $NH_4^+$ is the only form of DIN that can be significantly removed via sorption onto soil minerals. As with phosphorus, studies have attempted to increase $NH_4^+$ removal through soil amendment (Phillips 1998). However, as in natural systems, removals will be finite unless sorption sites are chemically or biologically regenerated. Nitrate, in contrast to $NH_4^+$ and DIP, is quite mobile and likely to be physically removed from BMPs only through groundwater flow. Therefore, $NO_3^-$ is a contaminant of concern in many aquifer source waters and may additionally impact receiving waters that receive substantial groundwater inputs.

According to O'Shea et al. (2002), data on phosphorus retention by stormwater BMPs often confusedly reflect a mix of short-term (e.g., plant uptake, exchangeable sorption) and longer-term (e.g., sedimentation, burial) removal processes. Monitoring data collected only during the growing season may overestimate capture since much of the nutrient load incorporated into plant or microbial biomass during growth phases will be released upon die back. Similarly, phosphorus removed via sorption processes can be rapidly released into the water column upon changing (e.g., seasonal) water column or sediment conditions (e.g., redox potential or pH). In addition to seasonal differences, phosphorus removal in newly constructed BMPs can also significantly decline after the first few years of treatment due to the saturation of sorption sites. A variety of soil amendments have been examined as a means to increase the sorption capacity of wetland or filtration media.

O'Shea also reports that in the absence of harvesting measures aimed at removing organically bound nutrients (and generally considered not cost-effective), burial in the sediment layer is considered one of the only processes capable of providing substantial and long-term phosphorus retention. Burial can occurs by several means including the settling of incoming solids and their associated phosphorus load, the formation of chemical precipitates, sorption onto solids or biofilms in the water column or sediment layer, and the accumulation and burial of refractory organic matter. However, not all of these processes can be considered irreversible. For example, settled phosphorus can become re-suspended by mixing events, often in a more bioavailable form due to diagenetic transformations. Other processes, such as sorption or precipitation, may provide only temporary detention, ultimately operating in reverse to release phosphorus into surface waters.

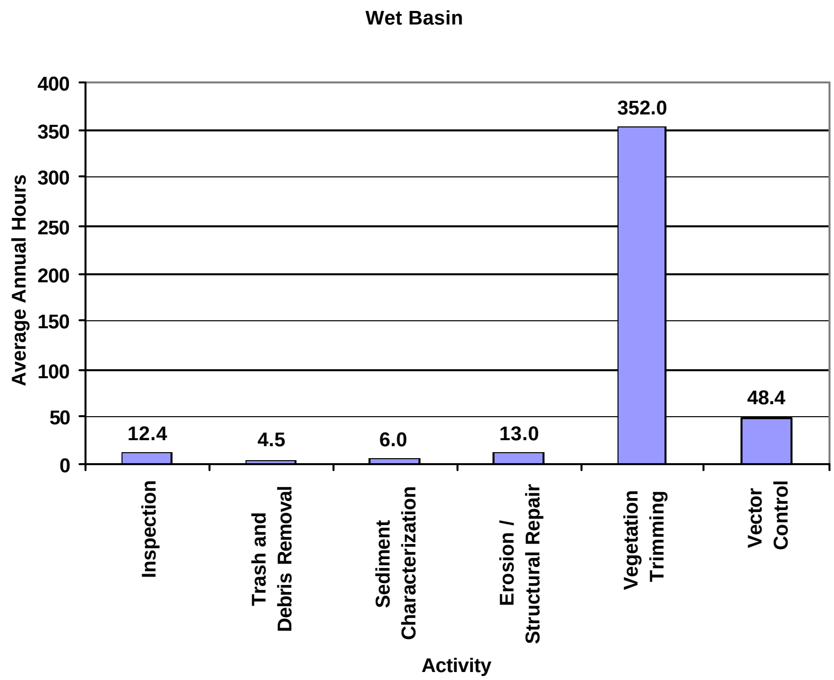
Nitrogen, like phosphorus, is present in receiving waters and surface runoff in a number of forms including dissolved and particulate, organic and inorganic species. Dissolved inorganic nitrogen (DIN) species, in particular ammonium ($NH_4^+$), are generally considered the most available for immediate assimilation by aquatic micro- and macrobiota. However, like phosphorus, biotic uptake is generally considered to provide mainly short-term (i.e., seasonal), nitrogen removal with much of the stored nutrients released as dissolved (DON) and particulate (PON) organic species during fall/winter senescence.

Options for long-term removal of nitrogen include not only burial, as with phosphorus, but also the biologically mediated sequential reactions, ammonification (the mineralization of organic nitrogen into ammonia and ammonium ($NH_3/NH_4^+$), nitrification (the oxidation of $NH_3/NH_4^+$ to nitrate ($NO_3^-$) and denitrification (the anaerobic reduction of $NO_3^-$) that results in the gaseous end products nitric oxide (NO), nitrogen gas ($N_2$), and nitrous oxide ($N_2O$). Together with the pH-mediated formation of volatile $NH_3$, the formation and export of these gaseous species generally account for a significant portion of the nitrogen loss observed in receiving waters and treatment systems. Treatment systems often attempt to promote these reactions by providing the conditions and residence times necessary for each transformation.

#### E1-2.3.7 Maintenance

Maintenance requirements of wet ponds vary greatly depending on local regulations, aesthetic considerations, the quality of the baseflow that sustains the permanent pool, and climatic considerations. In southern California, vector concerns (mosquito breeding, primarily) was one of the major drivers for maintenance. Local vector control agencies required annual removal of the majority of the vegetation to allow access to the mosquito larvae by the mosquito fish (*Gambusia affinis*). This vegetation harvesting accounts for the vast majority of the 250 hrs/yr estimated for future maintenance activities at a fairly small facility that treated the runoff from only 4.2 acres (Caltrans, 2002). The breakdown of hours actually incurred by Caltrans in a very aggressive maintenance program associated with research study is shown in Figure E1-10. The vegetation at this site was particularly vigorous because of the high nutrient concentrations in the perennial baseflow (15.5 mg/L $NO_3$-N) and the mild climate, which permits growth year round. It should be noted that vegetation harvesting removes nutrients from the system, preventing recycling, and potentially improving the removal of nutrients from stormwater.

**Figure E1-10 Actual Maintenance Hours for a Wet Basin in Southern California**




Since wet ponds are often selected for their aesthetic considerations as well as pollutant removal, they are often sited in areas of high visibility. Consequently, floating litter and debris are removed more frequently than would be required simply to support proper functioning of the pond and outlet. This is one of the primary maintenance activities performed at the Central Market Pond located in Austin, Texas (Glick, 2001). In this type of setting, vegetation management in the area surrounding the pond can also contribute substantially to the overall maintenance requirements.

One normally thinks of sediment removal as one of the typical activities performed at stormwater BMPs. This activity does not normally constitute one of the major activities on an annual basis. At the concentrations of TSS observed in urban runoff from stable watersheds, sediment removal may only be required every 20 years or so. Because this activity is performed so infrequently, accurate costs for this activity are lacking

In addition to incorporating features into the pond design to minimize maintenance, some regular maintenance and inspection practices are needed.. Table E1-9 outlines these practices.

**Table E1-9 Typical Maintenance Activities for Wet Ponds**

| Activity | Schedule |
|---|---|
| If wetland components are included, inspect for invasive vegetation. | Semi-annually inspection |
| Inspect for damage.<br>Note signs of hydrocarbon build-up, and deal with appropriately.<br>Monitor for sediment accumulation in the facility and forebay. | Annual inspection |
| Collect litter. | Monthly |
| Examine to ensure that inlet and outlet devices are free of debris and operational.<br>Clean and remove debris from inlet and outlet structures | Quarterly, and following large storms |
| Repair undercut or eroded areas. | As needed maintenance |
| Mow side slopes. | Monthly maintenance |
| Manage and harvest wetland plants. | Annual maintenance (if needed) |
| Remove sediment from the forebay. | Post construction and then 5- to 7-year maintenance regime (or once 50% of forebay capacity has been lost) |
| Maintain fence. | As needed maintenance |
| Monitor sediment accumulations, and remove sediment when the pool volume has become reduced significantly or the pond becomes eutrophic. | 20-to 50-year maintenance (or when sediment accumulation is significant) |

Source: WMI, 1997

### E1-2.3.8 Cost

#### *E1-2.3.8.1 Construction Costs*

Wet ponds can be relatively inexpensive stormwater practices; however, the construction costs associated with these facilities vary considerably. Much of the variability in costs can be attributed to the degree to which the existing topography will support a wet pond, the complexity and amount of concrete required for the outlet structure, and whether it is installed as part of new construction or implemented as a retrofit of an existing storm drain system. Value of land and loss of space for property development may add to the cost of the drainage infrastructure, but are not included in the cost numbers reported below.

A recent study (Brown and Schueler, 1997) estimated the cost of a variety of stormwater management practices. The study resulted in the following cost equation, adjusting for inflation:

$$C = 24.5V^{0.705}$$

where:

C = Construction, design and permitting cost;
V = Volume in the pond to include the 10-year storm ($ft^3$).

Using this equation, typical construction costs are:

$45,700 for a 1,200 $m^3$(1 acre-foot) facility
$232,000 for a 12,000 $m^3$ (10 acre-foot) facility
$1,170,000 for a 120,000 $m^3$ (100 acre-foot) facility

In contrast, Caltrans (2002) reported spending over $448,000 for a pond with a total permanent pool plus water quality volume of only 1036 $m^3$ (0.8 ac.-ft.), while the City of Austin spent $584,000 (including design) for a pond with a permanent pool volume of 3,100 $m^3$ (2.5 ac.-ft.).

Ponds do not consume a large area relative to the contributing drainage area (typically 2–3 percent); however, these facilities are generally large relative to other BMPs. Other practices, such as filters or swales, may be "squeezed" into relatively unusable land, but ponds need a relatively large continuous area.

Typical costs for wet ponds range form $17.50 to $35 per cubic meter ($0.50 - $1.00 per $ft^3$) (CWP, 1998). Several studies have shown retrofitting a wet pond to a developed area may be 5-10 times the costs of constructing the same size pond in an undeveloped area. (USEPA, 1999).

*E1-2.3.8.2 Maintenance Costs*

For ponds, the annual cost of routine maintenance has typically been estimated at about 3 to 5 percent of the construction cost; however, the published literature is almost totally devoid of actual maintenance costs. Since ponds are long-lived facilities (typically longer than 20 years), major maintenance activities are unlikely to occur during a relatively short study.

Table E1-10 presents the annual maintenance costs estimated by Caltrans (2002) based on three years of monitoring of a pond treating runoff from 1.7 ha (4.2 acres). Labor hours were converted to cost assuming a fully burdened rate of $44/hr. As mentioned previously, almost all the activities under the heading "Maintenance" are associated with vegetation management. Consequently, maintenance costs may vary considerably at other sites depending on the aggressiveness of the vegetation management in that area. Total cost at this site falls within the 3-5 percent range reported above; however, the construction costs were much higher than those incurred at other locales.

**Table E1-10 Expected Annual Maintenance Costs**

| Activity | Labor Hours | Equipment & Material, $ | Cost, $ |
|---|---|---|---|
| Inspections | 8 | 0 | 352 |
| Maintenance | 262 | 375 | 11,903 |
| Vector Control | 12 | 0 | 744 |
| Administration | 3 | 0 | 133 |
| Materials | - | 4500 | 4,500 |
| **Total** | **285** | **$ 4,875** | **$17,631** |

### E1-2.3.9 Research Needs

Research needs relative to wet pond performance are described below.

#### *E1-2.3.9.1 Siting Criteria*

Siting criteria for wet ponds are well established, and no additional work in this area is recommended.

#### *E1-2.3.9.2 Design Guidelines*

The primary uncertainty related to current design guidelines is the size of the permanent pool relative to the design storm one wishes to treat. The size of the permanent pool is often recommended to be a multiple (one to three times) of the average storm for an area. There is little data to substantiate these recommendations and more research is recommended.

Other design issues needing research include:

- Ability to consistently achieve target detention times across all expected storm events.
- Ability to design for treatment volumes and detention times that best protect downstream channels.

#### *E1-2.3.9.3 Performance*

Much of the uncertainty related to performance is associated with the pollutant removal of the emergent vegetation. Some of the issues include:

- Need for more research into water quality performance, particularly long-term performance with mass balance Figure E1-s over several years. Some research on water quality performance of Scottish SUDS ponds has been done by MacDonald (1999), McLean (2001) and currently by Spitzer (Schlüter et al., 2002). Not enough is known about the impacts of the build-up in nutrient levels in vegetation on nutrient through-flow. The benefits of vegetation harvesting, and the best approach, needs to be addressed.

- Uptake of heavy metals.
- Degree to which pollutant removal rates are reduced during non-growing season.
- More research needed into rate of accumulation of sediments and increasing levels of toxicity. Some research into pollutant accumulation in pond sediments is still ongoing at the University of Edinburgh by Kate Heal.
- Relationship between pond performance and appearance and its perception locally.

*E1-2.3.9.4 Construction Guidelines and Cost*

Capital cost will vary widely depending on site and type of pond, but can probably be relatively easily established for each new pond by quantity surveyors. A related issue that needs more research is the influence of the pond on house prices and house salability. In addition research is needed into the effectiveness of different marginal arrangements in providing safety barriers.

*E1-2.3.9.5 Maintenance Guidelines and Cost*

Maintenance issues that warrant further research include:

- No information about the long term maintenance of ponds has been found.
- Impact of differing maintenance regimes on pond performance (hydraulics and water quality).
- Real maintenance needs associated with sediment management (as this is currently largely being ignored).

E1-2.3.10 References

Bannerman, R., and R. Dodds, 1992. Unpublished data. Bureau of Water Resources Management, Wisconsin Department of Natural Resources, Madison, WI.

Borden, R. C., J.L. Dorn, J.B. Stillman, and S.K. Liehr, 1996. *Evaluation of Ponds and Wetlands for Protection of Public Water Supplies.* Draft Report. Water Resources Research Institute of the University of North Carolina, Department of Civil Engineering, North Carolina State University, Raleigh, NC.

Brown, W., and T. Schueler, 1997. *The Economics of Stormwater BMPs in the Mid-Atlantic Region.* Prepared for the Chesapeake Research Consortium, Edgewater, MD, by the Center for Watershed Protection, Ellicott City, MD.

Caltrans, 2002. *Final Report: BMP Retrofit Pilot Program*, California Department of Transportation Report CTSW-RT-01-050, Sacramento, CA (in press).

City of Austin, TX. 1991. *Design Guidelines for Water Quality Control Basins*. Public Works Department, Austin, TX.

City of Austin, TX, 1996. Evaluation of Non-Point Source Controls: A 319 Grant Project. Draft Water Quality Report Series, Public Works Department, Austin, TX.

Cullum, M. 1985. Stormwater Runoff Analysis at a Single Family Residential Site. Publication 85-1. University of Central Florida, Orlando, FL. pp. 247–256.

Dorman, M.E., J. Hartigan, R.F. Steg, and T. Quasebarth, 1989. *Retention, Detention and Overland Flow for Pollutant Removal From Highway Stormwater Runoff.* Vol. 1 Research Report. FHWA/RD 89/202. Federal Highway Administration, Washington, DC.

Driscoll, E.D., 1983. *Performance of Detention Basins for Control of Urban Runoff Quality.* Presented at the 1983 International Symposium on Urban Hydrology, Hydraulics and Sedimentation Control, University of Kentucky, Lexington, KY.

Emmerling-Dinovo, C., 1995. Stormwater Detention Basins and Residential Locational Decisions. *Water Resources Bulletin*, 31(3):515–52.

Gain, W.S., 1996. *The Effects of Flow Path Modification on Water Quality Constituent Retention in an Urban Stormwater Detention Pond and Wetland System.* Water Resources Investigations Report 95-4297. U.S. Geological Survey, Tallahassee, FL.

Galli, F., 1990. *Thermal Impacts Associated with Urbanization and Stormwater Best Management Practices.* Prepared for the Maryland Department of the Environment, Baltimore, MD, by the Metropolitan Council of Governments, Washington, DC.

Glick, Roger, 2001. Personal communication, City of Austin Watershed Protection Department, Austin, TX.

Holler, J.D., 1989. “Water Quality Efficiency of an Urban Commercial Wet Detention Storm Water Management System at Boynton Beach Mall in South Palm Beach County, FL,” *Florida Scientist* 52(1):48–57.

Holler, J.D., 1990. “Nonpoint Source Phosphorous Control By A Combination Wet Detention/Filtration Facility In Kissimmee, FL,” *Florida Scientist* 53(1):28–37.

Horner, R.R., J. Guedry, and M.H. Kortenhoff, 1990. *Improving the Cost Effectiveness of Highway Construction Site Erosion and Pollution Control.* Final Report. Washington State Transportation Commission, Olympia, WA.

Kantrowitz, I., and W. Woodham, 1995. *Efficiency of a Stormwater Detention Pond in Reducing Loads of Chemical and Physical Constituents in Urban Streamflow, Pinellas County, Florida.* Water Resources Investigations Report 94-4217. U.S. Geological Survey, Tallahassee, FL.

Martin, E., 1988. “Effectiveness Of An Urban Runoff Detention Pond/Wetland System,” *Journal of Environmental Engineering* 114(4):810–827.

Maryland Department of the Environment (MDE). 2000. *Maryland Stormwater Design Manual.* [http://www.mde.state.md.us/environment/wma/stormwatermanual].

O’Shea, M., Borst, M., Nietch, C., 2002. “The Role of Stormwater BMPs in Mitigating the Effects of Nutrient Overenrichment in the Urban Watershed,” in *Global Solutions for Urban Drainage*, Proceedings of the Ninth International Conference on Urban Drainage, September 8-13, 2002.

Oberts, G.L. 1994. “Performance Of Storm Water Ponds And Wetlands In Winter,” *Watershed Protection Techniques* 1(2):64–68.

Oberts, G.L., P.J. Wotzka, and J.A. Hartsoe, 1989. *The Water Quality Performance of Select Urban Runoff Treatment Systems.* Publication No. 590-89-062a. Prepared for the Legislative Commission on Minnesota Resources, Metropolitan Council, St. Paul, MN.

Oberts, G.L., and L. Wotzka, 1988. “The Water Quality Performance Of A Detention Basin Wetland Treatment System In An Urban Area,” *Nonpoint Source Pollution: Economy, Policy, Management and Appropriate Technology*. American Water Resources Association, Middleburg, VA.

Occoquan Watershed Monitoring Laboratory, 1983. Metropolitan Washington Urban Runoff Project. Final Report. Prepared for the Metropolitan Washington Council of Governments, Washington, DC, by the Occoquan Watershed Monitoring Laboratory, Manassas, VA.

Ontario Ministry of the Environment, 1991. *Stormwater Quality Best Management Practices.* Marshall Macklin Monaghan Limited, Toronto, Ontario.

Saunders, G. and M. Gilroy. 1997. *Treatment of Nonpoint Source Pollution with Wetland/Aquatic Ecosystem Best Management Practices.* Texas Water Development Board, Lower Colorado River Authority, Austin, TX.

Schueler, T., 1997a. "Comparative Pollutant Removal Capability of Urban BMPs: A Reanalysis," *Watershed Protection Techniques* 2(4):515–520.

Schueler, T. 1997b, "Influence of Groundwater on Performance of Storm Water Ponds in Florida,: *Watershed Protection Techniques* 2(4):525–528.

Urbonas, B., J. Carlson, and B. Vang, 1994. Joint Pond-Wetland System in Colorado. Denver Urban Drainage and Flood Control District, Denver, CO.

Urbonas, B., et al., 1992. *Urban Strom Drainage Criteria Manual, Volume 3 – Best Management Practices, Stormwater Quality*, Urban Drainage and Flood Control District, Denver, CO.

U.S. Environmental Protection Agency (USEPA), 1995. *Economic Benefits of Runoff Controls*. U.S. Environmental Protection Agency, Office of Wetlands, Oceans, and Watersheds, Washington, DC.

Watershed Management Institute (WMI), 1997. *Operation, Maintenance, and Management of Stormwater Management Systems.* Prepared for U.S. Environmental Protection Agency, Office of Water, Washington, DC, by the Watershed Management Institute, Ingleside, MD.

Wu, J., 1989. *Evaluation of Detention Basin Performance in the Piedmont Region of North Carolina.* Report No. 89-248. North Carolina Water Resources Research Institute, Raleigh, NC.

Yousef, Y., M. Wanielista, and H. Harper, 1986. "Design and Effectiveness of Urban Retention Basins," in *Urban Runoff Quality—Impact and Quality Enhancement Technology.* B. Urbonas and L.A. Roesner (eds.). American Society of Civil Engineering, New York, New York. pp. 338–350.

**Additional Reading**

Behrendt, L., 2002. *Diversity and Steadiness of Aquatic Macrophytes in Retention Ponds of Central Scotland as Part of "Sustainable Urban Drainage Systems" and Consideration of Heavy Metal Concentrations in Plant Tissues and Marginal Sediment.* MSc Thesis, Hochschule Zittau / Goerlitz.

Brix, H., 1997. "Do Macrophytes Play a Role in Constructed Treatment Wetlands?" *Water Science Technology*, 35 pp 11-17.

Foo, E.K.C., 2000. *Coliform Removal From Urban Runoff By Passive Detention Systems.* Part of MSc Thesis, University of Edinburgh.

Gumbricht, T., 1993. "Nutrient Removal Processes in Freshwater Submersed Macrophytes Systems," *Ecological Engineering*, 3, pp 1-30.

Jefferies, C., Aitken, A., McLean, N. MacDonald, K., McKissock, G., 1999. "Assessing the Performance of Urban BMPs in Scotland," *Water Science Technology*, 39, 12, pp 123-131.

Knight, R.L. 1997. "Wildlife Habitat and Public Use Benefits of Treatment Wetlands," *Water Science Technology*, 35, 5, pp 11-17.

Macdonald, K., Jefferies, C. and Dring D., (1999). *An Assessment of the Efficiency of a Stormwater Runoff Pond in the Scottish Climate.* IAWQ 8th International Conference on Urban Storm Drainage. 30 August - 3 September 1999, Sydney, Australia

McKissock G., D'Arcy, B.J., Jefferies C., 2001. *Sustainable Urban Drainage: A Case Study.* Paper presented on Novatech 2001-Innovative technologies in urban drainage conference proceedings, 25-27 June 2001, Lyon, France. GRAIE, Villeurbanne, Lyon, France.

McLean, N ., 2001. "A Study of the Effectiveness of Two SUDS Ponds Serving Highway Runof," in Jefferies, C., *SUDS Monitoring Programme – Report and Interim Outcomes,* Scotland and Northern Ireland Forum for Environmental Research, Report No SR(00)10 August 2001

Pond Action. 2000. *An Assessment of the Ecological Value of Sustainable Urban Drainage System Ponds in Scotland.* Pond Action, Oxford.

Roesner, L.A., Campbell, N., D'Arcy, B.J., 2001. *Master Planning Stormwater Management Facilities for the Dunfermline, Scotland Expansion Project.* Paper presented on Novatech 2001- Innovative technologies in urban drainage conference proceedings, 25-27 June 2001, Lyon, France. GRAIE, Villeurbanne, Lyon, France.

Schlüter, W., Spitzer A., Jefferies C., 2002. *Performance of 3 Sustainable Urban Drainage Systems in East Scotland*. 9th International Conference on Urban Drainage, Portland Oregon, September 2002.

SEPA (n.d.). *Protecting The Quality: A Guide To Sustainable Urban Drainage.* Scottish Environment Protection Agency, Edinburgh.

SEPA, 2000. *Ponds, Pools and Lochans: Guidance on Good Practice in the Management and Creation of Small Water Bodies in Scotland.* Habitat Enhancement Initiative, Scottish Environment Protection Agency, Edinburgh.

Stahre, P., 1999. *10 years of Experiences of Sustainable Stormwater Management in the City of Malmö, Sweden.* 8th International Conference on Urban Storm Drainage 30Aug – 3 Sept. 1999, Proceedings (vol 3), pp 1087-1097.

Spitzer, A., 2000. *Preliminary Results from the Hydrological and Water Quality Investigations at Duloch Park.* Paper Presented to SUDS Monitoring Workshop at Aberdeen University, September 26, 2000.

Spitzer, A., 2000. *Hydrological Performance of Two Retention Ponds at Duloch Park.* Scottish Universities SUDS Monitoring Programme, Report on First Phase of the Programme 1997 – 2001; Draft for Discussion in Workshop on 25 April 2001 at University of Abertay Dundee.

Spitzer, A. 2001. *Hydrological Investigations at Duloch Park – First Results.* Paper presented at Post Graduate Student Workshop of University of Abertay Dundee on 21 May 2001.

## E1-2.4 Vegetated Swales

### E1-2.4.1 Description

A vegetated swale is a broad, shallow channel with a dense stand of vegetation covering the side slopes and bottom. Swales can be natural or manmade, and are designed to trap particulate pollutants (suspended solids and trace metals), promote infiltration, and reduce the flow velocity of stormwater runoff. A typical example is shown in system, but are limited to the amount of runoff they can treat effectively. Therefore, swales are best suited for residential, industrial, and commercial areas with low flow and smaller populations.

As stormwater runoff flows through these vegetated channels, sedimentation is promoted by the relatively low velocities and shallow water depths. In addition, some uptake of nutrients by the vegetation may occur and a substantial amount of the runoff infiltrates into the underlying soils. Through geometric modifications and other features, the swale can be designed as both a treatment and conveyance system.

In cold or snowy climates, swales may serve a dual purpose by acting as both a snow storage/treatment and a stormwater management practice. This dual purpose is particularly relevant when swales are used to treat road runoff. If used for this purpose, swales should incorporate salt-tolerant vegetation, such as creeping bentgrass.

In arid or semi-arid climates, swales should be designed with drought-tolerant vegetation, such as buffalo grass. The value of vegetated practices for water quality management obviously needs to be weighed against the cost of the water needed to maintain them in these conditions.

Figure E1-11 and a schematic is given in **Error! Reference source not found.**. Vegetated swales can replace curbs, gutters and pipes in a stormwater conveyance system, but are limited to the amount of runoff they can treat effectively. Therefore, swales are best suited for residential, industrial, and commercial areas with low flow and smaller populations.

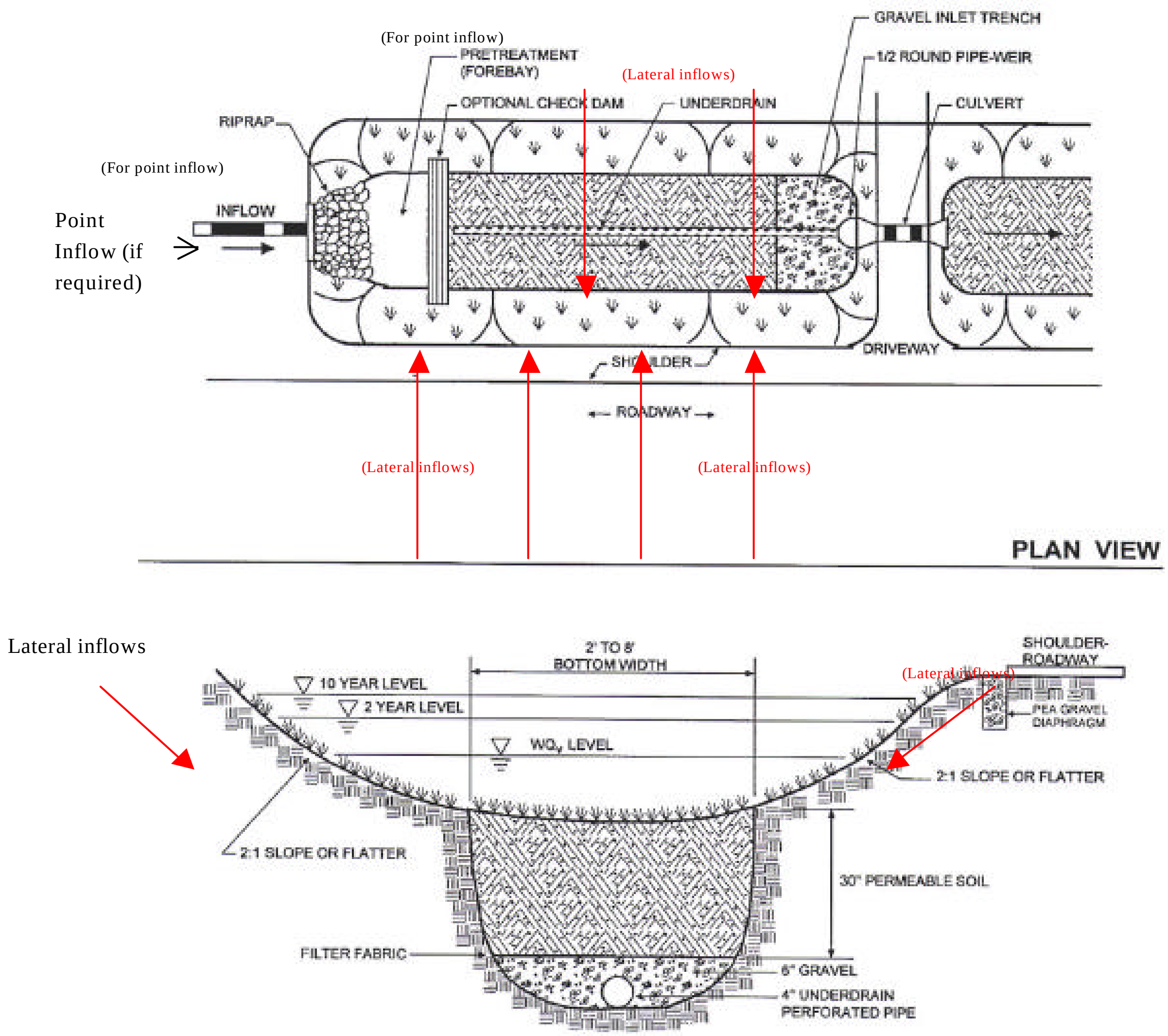
As stormwater runoff flows through these vegetated channels, sedimentation is promoted by the relatively low velocities and shallow water depths. In addition, some uptake of nutrients by the vegetation may occur and a substantial amount of the runoff infiltrates into the underlying soils. Through geometric modifications and other features, the swale can be designed as both a treatment and conveyance system.

In cold or snowy climates, swales may serve a dual purpose by acting as both a snow storage/treatment and a stormwater management practice. This dual purpose is particularly relevant when swales are used to treat road runoff. If used for this purpose, swales should incorporate salt-tolerant vegetation, such as creeping bentgrass.

In arid or semi-arid climates, swales should be designed with drought-tolerant vegetation, such as buffalo grass. The value of vegetated practices for water quality management obviously needs to be weighed against the cost of the water needed to maintain them in these conditions.

**Figure E1-11 A Vegetated Swale along I-15, San Diego County**

**Figure E1-12 Schematic of a Swale (MDE, 2000)**



### E1-2.4.2 Limitations

Grassed swales have some limitations, including the following:

- Grassed swales cannot treat a very large drainage area.
- If designed improperly (e.g., if longitudinal slope is too great), grassed channels will have very little pollutant removal.
- They are impractical in areas with very flat grades, steep topography, or wet or poorly drained soils.
- They are not effective and may even erode when flow volumes and/or velocities are high.
- Sufficient land may not be available for suitable swale designs to be incorporated.
- In some places in the US, their use is restricted by law. Local municipalities/ councils in the US and UK may require curb and gutter systems in residential areas.
- They are impractical in areas with erosive soils or where a dense vegetative cover is difficult to maintain.
- Infiltration through the swale may carry pollutants into local groundwater.

### E1-2.4.3 Advantages

- If properly designed, vegetated, and operated, swales can serve as an aesthetic, potentially inexpensive urban development or roadway drainage conveyance measure with significant collateral water quality benefits.
- Roadside ditches should be regarded as significant potential swale/buffer strip sites and should be utilized for this purpose whenever possible.
- The relatively narrow configuration could be amenable to its incorporation into site planning.
- Maintenance consists mainly of conventional landscaping and lawn measures.
- Infiltration and evapotranspiration reduce pollutant loadings and volume of runoff.

These vegetated conveyance systems can be subdivided into categories such as grassed channel, wet swale, and dry swale. Grassed channels would consist of vegetated channels not engineered specifically for water quality, while wet swales maintain a wetland assemblage due to high groundwater levels and frequent rain. In the Orange County area, only conventional dry swales would be appropriate for meeting stormwater permit requirements.

### E1-2.4.4 Siting Criteria

The suitability of a swale at a site will depend on land use, size of the area serviced, soil type, slope, imperviousness of the contributing watershed, and dimensions and slope of the swale system (Schueler et al., 1992). In general, swales can be used to serve areas of less than 10 acres, with slopes no greater than 6 percent. The seasonal high water table should be at least 4 feet below the surface. Use of natural topographic lows is encouraged and historic natural drainage courses should be considered, where appropriate (Young et al., 1996). Roadside ditches should be regarded as significant potential swale/buffer strip sites and should be utilized for this purpose whenever possible. However, the use of swales in areas subject to frequent roadside parking should be avoided, as this can cause over-compaction of the soil and deterioration of the surface (CIRIA, 2001).

Research in the Austin, Texas, area indicates that vegetated controls are effective at removing pollutants even when dormant (Barrett et al., 1998). Therefore, irrigation is not required to maintain growth during dry periods, but may be necessary only to prevent the vegetation from dying.

The topography of the site should permit the design of a channel with appropriate slope and cross-sectional area. Site topography may also dictate a need for additional structural controls. Recommendations for longitudinal slopes range between 2 and 6 percent. Flatter slopes can be used, if sufficient to provide adequate conveyance. Steep slopes increase flow velocity, decrease detention time, and may require energy dissipating and grade checks. Steep slopes also can be managed using a series of check dams to terrace the swale and reduce the slope to within acceptable limits. The use of check dams with swales also promotes infiltration and increases flow attenuation.

### E1-2.4.5 Design Guidelines

Swales can be used purely as a conveyance system to direct and convey runoff from the drained area to another stage of the surface water management system. Swales can also be designed for runoff attenuation, for treatment, and for disposal (using infiltration through the base of the swale) (CIRIA, 2001).

Almost all of the current guidelines for swales, both in the United States and the United Kingdom, are founded on the results of a single study conducted in the US (Seattle Metro and Washington Department of Ecology, 1992). This study is the basis for hydraulic residence time requirements, appropriate slopes, and maximum velocities for the water quality design storm. Most studies conducted subsequently were on swales designed according to these recommendations and generally confirm that substantial pollutant removal does occur in the recommended design, but do little to expand on whether similar performance might be observed in other configurations.

In the US, a trapezoidal cross-section is normally recommended (i.e., WSDOT, 1995); however, the main advantage seems to be the ease at calculating water depth for the design storm intensity. Barrett et al. (1998) demonstrated that when runoff enters the swale from the side, the majority of the pollutant removal occurs on the side slopes; consequently, other geometries which include wider, flatter sides may actually perform better.

Swales to be used for conveyance should be designed in accordance with standard hydraulic principles. Commonly recommended design guidelines are summarized in Table E1-11, with velocities and residence times calculated using Manning's Equation with an "n" value of 0.25 for the water quality design storm flow. An "n" of 0.02 is recommended for larger events.

**Table E1-11 Design Guidelines for Vegetated Swales**

| Design Criteria | Current Guidance/Recommendations |
|---|---|
| Hydraulic Residence Time (Swale Length) | • Most of the design guidelines adopted for swale design specify a minimum hydraulic residence time of 9 minutes. This criterion is based on the results of a single study conducted in Seattle, Washington (Seattle Metro and Washington Department of Ecology, 1992), and is not well supported. Analysis of the data collected in that study indicates that pollutant removal at a residence time of 5 minutes was not significantly different, although there is more variability in that data. |
| Planting | • Swales must be vegetated in order to provide adequate treatment of runoff. It is important to maximize water contact with vegetation and the soil surface.<br>• Pollutant removal efficiencies are not very dependent on the specific plants involved, so the vegetation should be selected with stabilization of the soils as the primary consideration. (Barrett, 1998; Caltrans, 2002)<br>• A fine, close growing, water resistant grass should be selected for use in vegetated swales. Guides to swale design often suggest a grass length; for example, 150 mm (0.5 ft), SEPA, (1999). Many design guidelines recommend that grass be frequently mowed to maintain dense coverage near the ground surface, and observational experience would suggest that short cut grass is most likely to be accepted in the UK. Recent research (Colwell et al., 2000), however, has shown mowing frequency or grass height has little or no effect on pollutant removal.<br>• The vegetation in the base of the swale should be maintained at or just above |

| Design Criteria | Current Guidance/Recommendations |
|---|---|
| | the maximum water level for the water quality storm. (CIRIA) |
| Side Slopes | • The side slopes should be no steeper than 3:1 (H:V) (WSDOT, 1995).<br>• The sides of the swale should not be steeper than 1 in 4, to allow for mowing and access for maintenance personnel. |
| Bottom Width | • The maximum bottom width is 2.6 m (10 feet) unless a dividing berm is provided (WSDOT, 1995). (2) This will discourage formation of gullies and ensure there is a consistent spread of flow across the swale. |
| Flow Depths | • The depth of flow should not exceed 10 cm (4 in) during a 2.5 cm / hr (1 in / hr) storm.<br>• The depth of flow should not exceed 0.1 m (.33 ft) (CIRIA, 2001)<br>• The flow depth should be less than the height of the grass to ensure filtration (CIRIA, 2000) or a grass swale's depth should not exceed one-third of the grass height in infrequently mowed swales, or one half of the grass height in regularly mowed swales (Horner et al, 1994) |
| Longitudinal Channel Slope | • The channel slope should be at least 1% (in wet climates) and no greater than 5% (WSDOT, 1995), although recent work (Colwell, 2001) suggests that stability is increased if the slope does not exceed 2.5%.<br>• The longitudinal bottom slope of all types of swale should be kept as level as possible, and ideally no greater than 2% (CIRIA, 2001)<br>• Steeper overall longitudinal slopes can be accommodated by the introduction of check dams spaced at intervals along the length of the swale. Criteria for designing the check dams are as follows (CIRIA, 2001):<br>1. They should be designed to retain the runoff upstream.<br>2. Interconnections between adjacent ponds should be designed so that the flow between them does not re-suspend settled material or cause local erosion.<br>3. Interconnections between adjacent sections should be designed to retain floating solids or surface films.<br>• Where slopes are greater than 6%, infiltration will not be effective and the system will only act as a conveyance device (CIRIA, 2001) |
| Flow Velocities | • Flow velocities for side-slope flows should be less than 0.3 m / s (1 ft / s) in order to promote filtration and settlement (CIRIA, 2001).<br>• The runoff velocity in the swale should not exceed 0.3 m / s (1 ft / s) at the design treatment volume (CIRIA).<br>• At full conveyance depth, the velocity should not exceed 1.5m / s (CIRIA, 2001). |
| Inlet Infrastructure | • If flow is to be introduced through curb cuts, place pavement slightly above the elevation of the vegetated areas. Curb cuts should be at least 12 inches (30 cm) wide to prevent clogging (WSDOT, 1995).<br>• Inflow could be via a low earth weir at the edge of the swale (CIRIA, 2001).<br>• Inflow could be via a riprap or concrete chute, although in this case erosion |

| Design Criteria | Current Guidance/Recommendations |
|---|---|
| | protection measures will be required to protect the surface of the swale (CIRIA, 2001).<br>• Any inlet infrastructure that directs the flow to the base of the swale bypasses the beneficial filtering effect of the side slopes. |
| Pre-treatment | • Pretreatment is required to protect the swale. For pipe inlets, 0.1 in (.25 cm) per contributing acre should be temporally stored behind a check dam. For lateral inflows gentle slope or a pea gravel diaphragm can be used (Schueller, 1997). |
| Subsoil | • It is necessary to ensure that the design soil characteristics adequately represent conditions across the full length of the swale. Where additional ground investigation is not available, trial pits should be dug at 25 m (82 ft) intervals (CIRIA, 2001).<br>• It is often necessary to modify the parent soils to improve the infiltration rate. Dry swales will have a prepared soil filter bed that is composed of 50% sand 50% silt loam. Swale filter beds are drained by a longitudinal perforated pipe to keep swale dry after storm events (Schueler,1997).<br>• The underdrain system should be composed of a 6 in (15 cm) gravel bed with a 4 in (10 cm) PVC pipe.<br>• The soils used to finish the swale slopes should be suitably fertile, porous and of sufficient depth to ensure healthy vegetation growth (CIRIA, 2001). |
| Water Table | • Geotechnical tests must be performed to determine the location of the water table. If the water table is within 2 feet (0.6 m) of the planned swale bottom, a dry swale is not feasible (Schueler, 1997).<br>• If there is a risk of a swale conveying potential polluted water, and the groundwater requires protection, an impermeable liner should be incorporated in the design (CIRIA, 2001). |
| Shape | • The base of the swale should be flat, with a smooth transition between the base and the sides (CIRIA, 2001). |

### E1-2.4.6 Performance

The literature suggests that vegetated swales represent a practical and potentially effective technique for controlling urban runoff quality. It is known that check dams, slight slopes, permeable soils, dense grass cover, increased contact time, and small storm events all contribute to successful pollutant removal by the swale system. Factors decreasing the effectiveness of swales include compacted soils, short runoff contact time, large storm events, frozen ground, short grass heights, steep slopes, and high runoff velocities and discharge rates.

Vegetated swales and filter strips have not been accepted widely as primary controls for the treatment of stormwater runoff. This is mainly the result of the wide range of pollutant removals reported for vegetative controls in various studies (Schueler et al., 1992; Young et al., 1996; USEPA, 1983). Consequently, a lack of confidence emerged among regulatory agencies that vegetative controls could provide reliable and consistent removal of pollutants in stormwater. Most design manuals recommend vegetative controls only for pretreatment to reduce sediment loading to filtration systems or other structural stormwater controls. Unfortunately, many of the studies in which lower removal efficiencies were observed were not well designed and significant removal of the pollutants occurred before the runoff entered the test sections that were monitored.

One example is the evaluation of a grassy swale in Austin, Texas, by Welborn and Veenhuis (1988). The authors reported low or negative removals for many stormwater constituents; however, the runoff had traveled through a grassy swale for more than 60 m (200 ft) *before* reaching the influent to the site that was monitored. The median influent suspended solids concentration was less than 20 mg/L, even though the drainage was derived from an area of medium density townhouses. Unpublished data collected by the City of Austin indicate that TSS concentrations in stormwater runoff from this type of land use are typically above 100 mg/L. The low influent concentration indicates that most of the removal occurred before entering the test section; therefore, the removals reported understate the potential improvement in water quality resulting from use of vegetative controls.

Kaighn and Yu (1996) recognized that the quality of highway runoff entering two test swales was considerably better than that observed at an edge of pavement site located nearby. For example, average TSS concentrations in runoff entering the swale were 38.7 and 32.8 mg/L, while runoff sampled from the pavement nearby had a TSS concentration of 112.9 mg/L. Additional monitoring indicated that much of the removal occurred in the vegetated filter strip crossed by the runoff before entering the swale test section.

Dorman et al. (1996) analyzed the performance of three vegetated channels for treating highway runoff and reported TSS removal efficiencies ranging from 98 percent to negative 7 (-7) percent. Operational problems and erosion of the channel were reported at the Virginia site. The influent TSS concentrations were as low as 8 mg/L at the Maryland site, and the average was less than 30 mg/L. The third site in Florida was extremely effective in removing suspended solids. The wide range of reported removal efficiencies reinforces the belief that small changes in channel characteristics cause large changes in pollutant removal effectiveness; however, only the data from the Florida site is indicative of the pollutant removal that might be expected in grassy channels.

The stormwater discharged from the vegetated controls in many of these studies contained 20 mg/L or less of TSS, which is well below the concentrations that would be expected based on the land use in the watershed. However, the impression conveyed by these published reports is that the vegetative controls are unreliable or worse, do nothing at all.

In the studies described above, the monitored vegetative controls generally operated to polish the quality of the discharge rather than as the primary treatment device. A more efficient and consistent performance would be expected when the concentrations in the influent to the controls are similar to what might be encountered in untreated urban or highway runoff. The TSS concentrations might range from 100 to 200 mg/L, which is almost an order of magnitude higher than the influent concentrations reported in the cited studies.

Although TSS removal may be substantial, performance for other constituents may be less. A project in Durham, NC, monitored the performance of a carefully designed artificial swale that received runoff from a commercial parking lot. The project tracked 11 storms and concluded that particulate concentrations of heavy metals (Cu, Pb, Zn, and Cd) were reduced by approximately 50 percent. However, the swale proved largely ineffective for removing soluble nutrients.

Caltrans (2002) conducted a study of the efficiency of six swales located in Southern California, and the detailed results of this study are presented in Table E1-12. The column entitled “Significance” is the probability that the mean influent and effluent concentrations are not significantly different, based on an ANOVA. At the test sites, approximately 50 percent of the runoff infiltrated, so the total load reduction is much greater than the concentration reduction shown in Table E1-12.

A summary of the results of other studies have been conducted on a variety of grassed channels is presented in Table E1-13. The data confirm the relatively high removal rates for some pollutants, but negative removals for bacteria, and fair performance for nutrients.
While it is difficult to distinguish between different designs based on the small amount of available data, grassed channels generally have poorer removal rates than wet and dry swales, although some swales appear to export soluble phosphorus (Harper, 1988; Koon, 1995). It is not clear why swales export bacteria. One explanation is that bacteria thrive in the warm swale soils.

Researchers in Sweden have tested the performance of laboratory constructed swales and working swales using a standardized runoff event simulation procedure. Backstrom’s (2001) studies focused on the removal of suspended solids, and demonstrated an average of 79 to 98 percent removal under different vegetative conditions.

Figure E1-13 through Figure E1-17 provide a summary of the tabulated results. From the results, it appears that standard drainage channels have a much more variable performance than designed swales, and their removal efficiencies are therefore not analyzed further.

**Table E1-12 Performance of Vegetated Swales (Caltrans, 2002)**

| Constituent | Mean EMC | | Removal, % | Significance, P |
|---|---|---|---|---|
| | Influent mg/L | Effluent mg/L | | |
| TSS | 94 | 47 | 49 | 0.002 |
| $NO_3$-N | 1.22 | 0.89 | 27 | 0.147 |
| TKN | 3.43 | 2.36 | 31 | 0.907 |
| Total N[a] | 4.64 | 3.24 | 30 | - |
| Ortho-phosphate | 0.13 | 0.40 | -218 | <0.000 |
| Phosphorus | 0.26 | 0.53 | -106 | 0.001 |
| Total Cu | 0.049 | 0.019 | 63 | <0.000 |
| Total Pb | 0.099 | 0.031 | 68 | 0.075 |
| Total Zn | 0.349 | 0.079 | 77 | <0.000 |
| Dissolved Cu | 0.024 | 0.012 | 49 | 0.067 |
| Dissolved Pb | 0.018 | 0.007 | 57 | 0.081 |
| Dissolved Zn | 0.170 | 0.045 | 74 | <0.000 |
| TPH-Oil [b] | 3.5 | 1.7 | 51 | 0.107 |
| TPH-Gasoline [b] | <0.05[c] | <0.05[c] | NA | - |
| TPH-Diesel [b] | 1.3 | 0.4 | 69 | 0.156 |
| Fecal Coliform [b] | 12,300 MPN / 100mL | 16,000 MPN / 100mL | -30 | 0.707 |

[a] Considered to be sum of $NO_3$-N and TKN-N.

[b] TPH and Coliform are collected by grab method and may not accurately reflect removal.

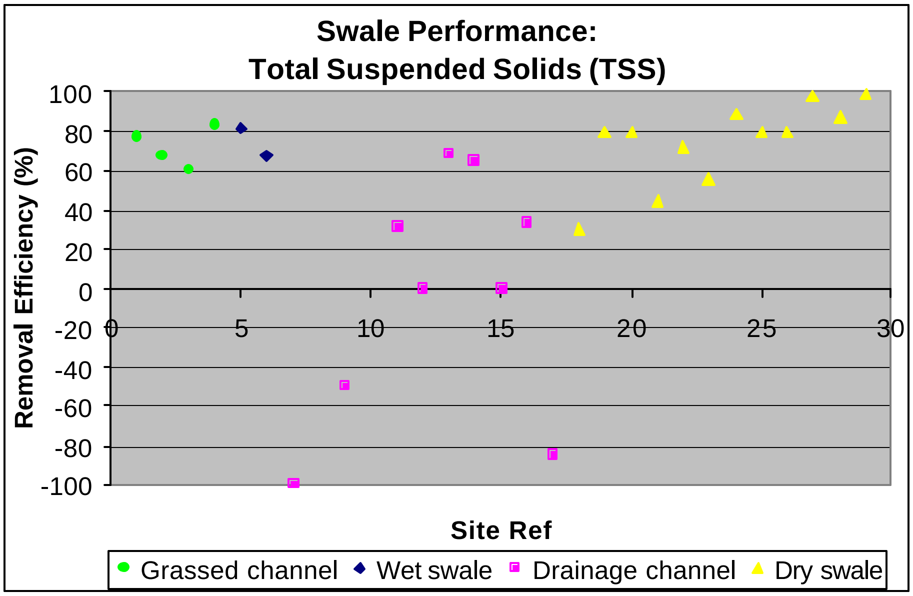
[c] Equals value of reporting limit.

**Table E1-13 Grassed Swale Pollutant Removal Efficiency Data**

| Removal Efficiencies (% Removal) | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Study** | **TSS** | **TP** | **TN** | **$NO_3$** | **Metals** | **Bacteria** | **Type** |
| Caltrans 2002 | 77 | 8 | 67 | 66 | 83-90 | -33 | |
| Goldberg 1993 | 67.8 | 4.5 | - | 31.4 | 42–62 | -100 | grassed channel |
| Seattle Metro and Washington Department of Ecology 1992 | 60 | 45 | - | -25 | 2–16 | -25 | grassed channel |
| Seattle Metro and Washington Department of Ecology, 1992 | 83 | 29 | - | -25 | 46–73 | -25 | grassed channel |
| Wang et al., 1981 | 80 | - | - | - | 70–80 | - | dry swale |
| Dorman et al., 1989 | 98 | 18 | - | 45 | 37–81 | - | dry swale |
| Harper, 1988 | 87 | 83 | 84 | 80 | 88–90 | - | dry swale |
| Kercher et al., 1983 | 99 | 99 | 99 | 99 | 99 | - | dry swale |
| Harper, 1988 | 81 | 17 | 40 | 52 | 37–69 | - | wet swale |
| Koon, 1995 | 67 | 39 | - | 9 | -35 to 6 | - | wet swale |
| Occoquan Watershed Monitoring Lab, 1983 | -100 | -100 | -100 | - | -100 | - | drainage channel |
| Yousef et al., 1985 | - | 8 | 13 | 11 | 14–29 | - | drainage channel |
| Occoquan Watershed Monitoring Lab, 1983 | -50 | -9.1 | -18.2 | - | -100 | - | drainage channel |
| Yousef et al., 1985 | - | -19.5 | 8 | 2 | 41–90 | - | drainage channel |
| Occoquan Watershed Monitoring Lab, 1983 | 31 | -23 | 36.5 | - | -100 to 33 | - | drainage channel |
| Welborn and Veenhuis, 1987 | 0 | -25 | -25 | -25 | 0 | - | drainage channel |
| Yu et al., 1993 | 68 | 60 | - | - | 74 | - | drainage channel |

| Removal Efficiencies (% Removal) | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Study** | **TSS** | **TP** | **TN** | **$NO_3$** | **Metals** | **Bacteria** | **Type** |
| Dorman et al., 1989 | 65 | 41 | - | 11 | 14-55 | - | drainage channel |
| Pitt and McLean, 1986 | 0 | - | 0 | - | 0 | 0 | drainage channel |
| Oakland, 1983 | 33 | -25 | - | - | 20–58 | 0 | drainage channel |
| Dorman et al., 1989 | -85 | 12 | - | -100 | 14–88 | - | drainage channel |
| Lawrence et al., 1997 | 20-40 | 20-40 | 20-40 | - | 0-20 | 20-40 | dry swale |
| Roesner et al., 1997 | 80 | 40 | 40 | 40 | 50-75 | - | dry swale |
| Honeyman et al., 2001a | 80 | | | | | | dry swale |
| Honeyman et al., 2001b | 44 | | | | | | dry swale |
| MacDonald, 2001a | 72 | | 100 | | | | dry swale |
| MacDonald, 2001b | 55 | 33 | -42 | | 43 - -423 | | dry swale |
| Backstrom, 2001 | 79-98 | | | | | | Laboratory experiment |
| Roesner et al., 1997 | 80 | 40 | 40 | 40 | 50-75 | - | dry swale |

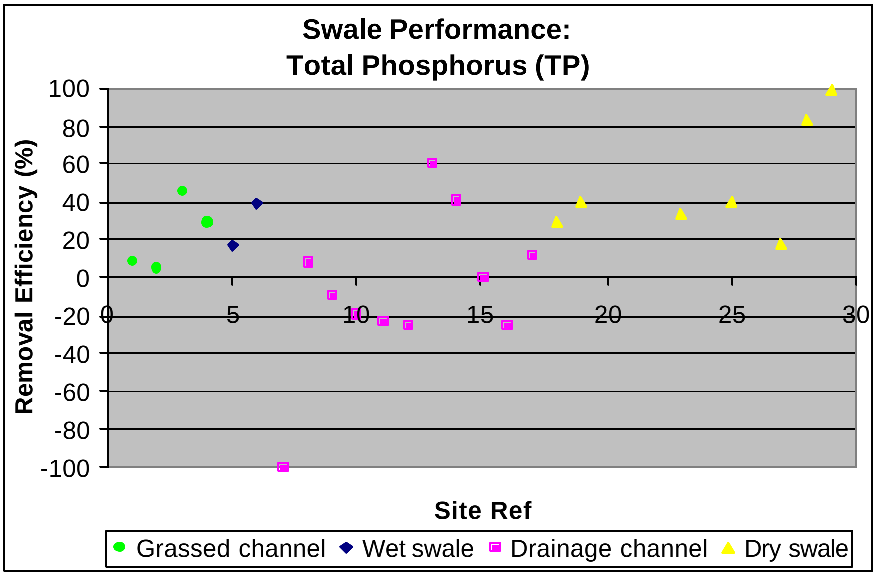
**Figure E1-13 Swale Performance for Removal of TSS**




Mean performance levels (in terms of efficiency of removal of total suspended solids) are as follows:

- Grassed channels: 72%
- Wet swales: 74%
- Dry swales: 74%

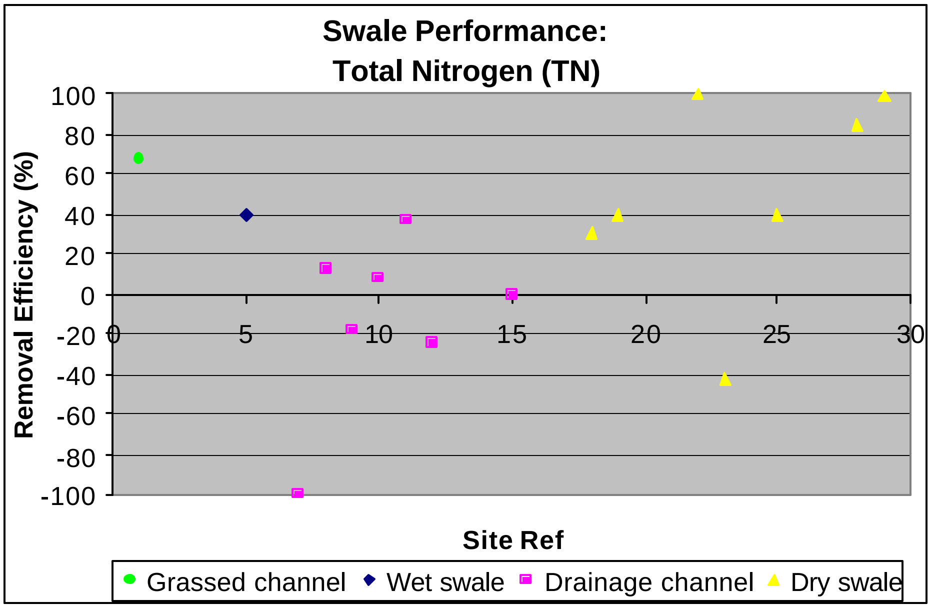
**Figure E1-14 Swale Performance for Removal of TP**




Mean performance levels (in terms of efficiency of removal of total phosphorous) are as follows:

- Grassed channels: 22%
- Wet swales: 28%
- Dry swales: 49%

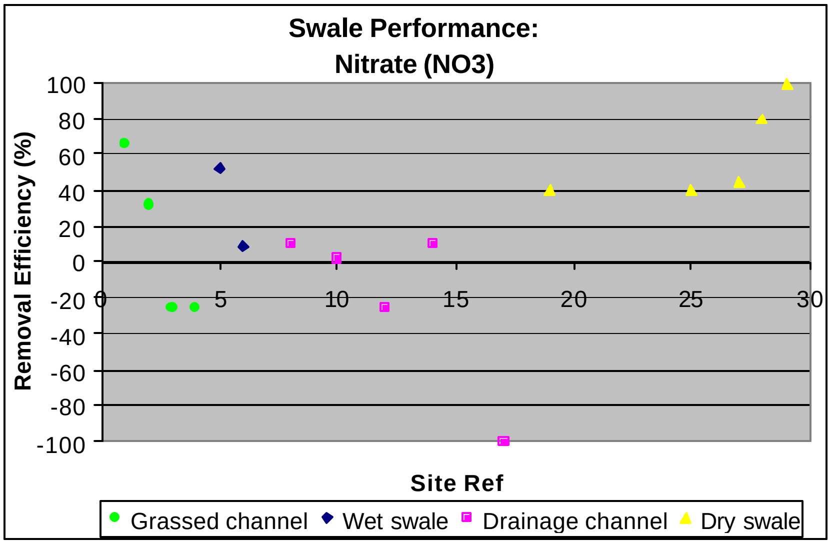
**Figure E1-15 Swale Performance for Removal of TN**




Mean performance levels (in terms of efficiency of removal of total nitrogen) are as follows:

- Grassed channels: 67% (based on 1 observation only)
- Wet swales: 40% (based on 1 observation only)
- Dry swales: 50%

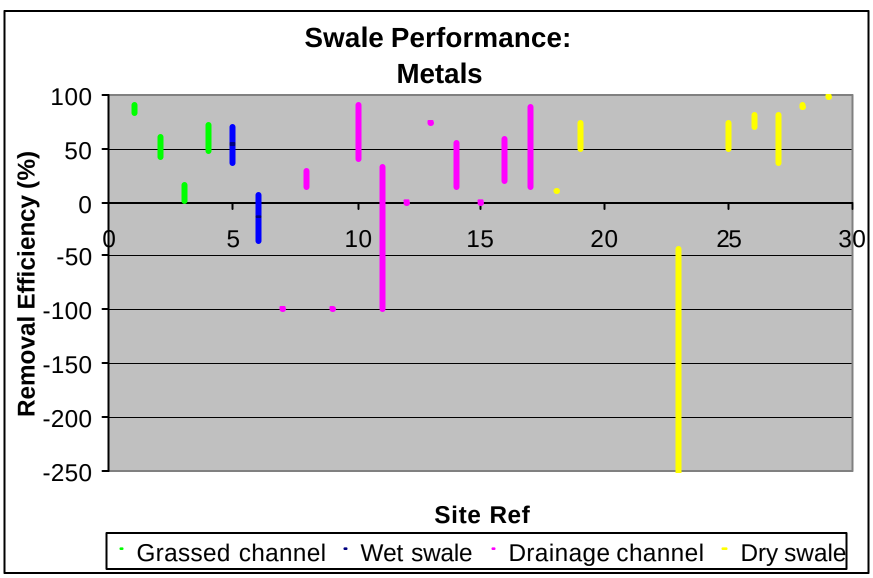
**Figure E1-16 Swale Performance for Removal of $NO_3$**




Mean performance levels (in terms of efficiency of removal of nitrites) are as follows:

- Grassed channels: 12%
- Wet swales: 31%
- Dry swales: 61%

**Figure E1-17 Swale Performance for Removal of Heavy Metals**




This graph shows high levels of variability across all swale types.

The effectiveness of vegetated swales can be enhanced by adding check dams at approximately 17 meter (50 foot) increments along their length (See Figure E1-7-1). These dams maximize the retention time within the swale, decrease flow velocities, and promote particulate settling. Finally, the incorporation of vegetated filter strips parallel to the top of the channel banks can help to treat sheet flows entering the swale.

#### E1-2.4.7 Maintenance

The useful life of a vegetated swale system is directly proportional to its maintenance frequency. The maintenance objectives for vegetated swale systems include keeping up the hydraulic and removal efficiency of the channel and maintaining a dense, healthy grass cover.

Maintenance activities should include periodic mowing (with grass never cut shorter than the design flow depth); weed control; watering during drought conditions, if needed to keep the vegetation alive; reseeding of bare areas; and clearing of debris and blockages. The frequency of mowing will be largely dependent on the species of grass planted but is likely to be necessary at least 2 to 3 times per year. Mowing and other maintenance equipment should not damage or excessively consolidate the surface (CIRIA, 2001). Cuttings should be removed from the channel and disposed in a local composting facility. Accumulated sediment should also be removed manually to avoid concentrated flows in the swale. The application of fertilizers and pesticides should be minimized.

Another aspect of a good maintenance plan is repairing damaged areas within a channel. For example, if the channel develops ruts or holes, it should be repaired utilizing a suitable soil that is properly tamped and seeded. The grass cover should be thick; if it is not, reseed as necessary. In the UK it is recommended that any standing water removed during the maintenance operation must be disposed to a sanitary sewer at an approved discharge location. Residuals (e.g., silt, grass cuttings) must be disposed in accordance with local or state requirements. Typical maintenance activities are summarized in Table E1-14.

#### E1-2.4.8 Cost

##### *E1-2.4.8.1 Construction Costs*

Little data are available to estimate the difference in cost between various swale designs. One study (SWRPC, 1991) estimated the construction cost of grassed channels at approximately $2.70 / m$^3$ ($0.25 per ft$^2$). This price does not include design costs or contingencies. Brown and Schueler (1997) estimate these costs at approximately 32 percent of construction costs for most stormwater management practices. For swales, however, these costs would probably be

**Table E1-14 Typical Maintenance Activities for Grassed Swales**

| Activity | Schedule |
|---|---|
| • Inspect pea gravel diaphragm for clogging and correct the problem.<br>• Inspect grass along side slopes for erosion and formation of rills or gullies and correct.<br>• Remove trash and debris accumulated in the inflow forebay.<br>• Inspect and correct erosion problems in the sand / soil bed of dry swales.<br>• Based on inspection, plant an alternative grass species, if the original grass cover has not been successfully established.<br>• Replant wetland species (for wet swale), if not sufficiently established. | Annually (semi-annually the first year) |
| • Rototill or cultivate the surface of the sand / soil bed of dry swales, if the swale does not draw down within 48 hours.<br>• Remove sediment build-up within the bottom of the swale once it has accumulated to 25% of the original design volume and scarify to encourage vegetation redevelopment or replant as necessary. | As needed (infrequent) |
| • Mow grass as needed for safety, aesthetic or other purposes. | As needed |

Source: Adapted from CWP, 1996

significantly higher, since the construction costs are so low compared with other practices. A more realistic estimate would be a total cost of approximately \$5.40 / $m^3$ (\$0.50 per $ft^2$), which compares favorably with other stormwater management practices.

Table E1-15 presents the construction costs compiled by Caltrans (2001) from a number of locations across the US, while Table E1-16 presents unit costs estimated by SWRPC (1991).

**Table E1-15 US Swale Construction Costs**

| Entity | Tributary Area (acres) | Water Quality Volume ($ft^3$) | Adjusted Total Cost, $ | Adjusted Total Cost per Acre Treated, $ |
|---|---|---|---|---|
| Maryland and Virginia CWP | 80.00 | - | 3,561 | 45 |
| Maryland and Virginia CWP | 5.00 | - | 18,932 | 3,786 |
| Maryland and Virginia CWP | 0.87 | 6,778 | 18,089 | 20,792 |
| ODOT | 1.17 | - | 41,736 | 35,672 |
| Caltrans | 2.40 | 6,916 | 136,822 | 57,009 |
| Caltrans | 2.30 | 6,650 | 140,006 | 60,872 |
| Caltrans | 0.40 | 1,742 | 31,992 | 79,979 |
| Caltrans | 0.70 | 2,614 | 70,138 | 100,197 |
| Caltrans | 0.70 | 2,178 | 76,179 | 108,827 |
| Caltrans | 0.50 | 1,742 | 125,488 | 250,977 |

### Table E1-16 Swale Cost Estimate (SWRPC, 1991)

| Component | Unit | Extent | Unit Cost | | | Total Cost | | |
|---|---|---|---|---|---|---|---|---|
| | | | Low, $ | Moderate, $ | High, $ | Low, $ | Moderate, $ | High, $ |
| Mobilization / Demobilization-Light | Swale | 1 | 107 | 274 | 441 | 107 | 274 | 441 |
| Site Preparation | | | | | | | | |
| Clearing[b]…………… | | | | | | | | |
| Grubbing[c]………… | Acre | 0.5 | 2,200 | 3800 | 5400 | 1,100 | 1,900 | 2,700 |
| General | Acre | 0.25 | 3,800 | 5200 | 6600 | 950 | 1,300 | 1,650 |
| Excavation[d]……….. | $Yd^3$ | 372 | 2.10 | 3.70 | 5.30 | 781 | 1,376 | 1,972 |
| Level and Till[e]…….. | $Yd^2$ | 1,210 | 0.20 | 0.35 | 0.50 | 242 | 424 | 605 |
| Sites Development | | | | | | | | |
| Salvaged Topsoil | | | | | | | | |
| Seed and Mulch[f] | $Yd^2$ | 1,210 | 0.40 | 1.00 | 1.60 | 484 | 1,210 | 1,936 |
| Sod[g] | $Yd^2$ | 1,210 | 1.20 | 2.40 | 3.60 | 1,452 | 2,904 | 4,356 |
| Subtotal | -- | -- | -- | -- | -- | 5,116 | 9,388 | 13,660 |
| Contingencies | Swale | 1 | 25% | 25% | 25% | 1,279 | 2,347 | 3,415 |
| Total | -- | -- | -- | -- | -- | 6,395 | 11,735 | 17,075 |

Note: Mobilization / demobilization refers to the organization and planning involved in establishing a vegetative swale.

[a] Swale has a bottom wi dth of 1.0 foot, a top width of 10 feet with 1:3 side slopes, and a 1,000 foot length.

[b] Area cleared = (top width + 10 feet) x swale length.

[c] Area grubbed = (top width x swale length).

[d] Volume excavated = (0.67 x top width x swale depth) x swale length (parabolic cross-section).

[e] Area tilled = $\left(\text{top width} + \frac{8(\text{swale depth}^2)}{3(\text{top width})}\right)$ x swale length (parabolic cross-section).

[F] Area needed = area cleared x 0.5.

[g] Area sodded = area cleared x 0.5.

*E1-2.4.8.2 Maintenance Costs*

Table E1-17 presents the expected annual maintenance cost for a swale with a tributary area of approximately 2 ha (5 ac.) as estimated by Caltrans (2002). Essentially, all the activities are related to vegetation management; consequently, no special training is required for maintenance personnel.

**Table E1-17 Expected Annual Maintenance Costs for Swales**

| Activity | Labor Hours | Equipment & Material, $ | Cost, $ |
|---|---|---|---|
| Inspections | 1 | 0 | 44 |
| Maintenance | 47 | 182 | 2,250 |
| Vector Control | 0 | 0 | 0 |
| Administration | 3 | 0 | 132 |
| Materials | - | 310 | 310 |
| **Total** | **51** | **$492** | **$2,736** |

Estimated maintenance costs for two sizes of swales, as presented by SEWRPC (1991), are presented in Table E1-18. Annual cost for maintaining vegetated swales is approximately $1.90 per linear meter for a 0.5 m (1.5 ft) deep channel.

E1-2.4.9 Research Needs

*E1-2.4.9.1 Siting Criteria*

Siting criteria for swales are well established and no additional work in this area is recommended.

*E1-2.4.9.2 Design Guidelines*

There are a number of uncertainties in the specific effect of several commonly adopted design guidelines, including residence time and side slopes. Clearly, a longer residence time allows more of the finer particles to settle out; however, there is likely an irreducible minimum concentration effluent concentration that can be obtained. Many requirements are based on the need to maintain channel stability during larger storms; consequently evaluation of minor differences in slope or length is a low priority. As with many other devices, a hydrological analysis of the potential changes in storm frequency and intensity would be useful for establishing the long-term viability of current designs.

**Table E1-18 Estimated Maintenance Costs SEWRPC, 1991**

| Component | Unit Cost | Swale Size (depth and top width) | | Comment |
|---|---|---|---|---|
| | | 0.5-ft depth, 1-ft bottom width, 10-ft top width | 3-ft depth, 3-ft bottom width, 21-ft top width | |
| Lawn mowing | $0.85 / 1000 $ft^2$ / mowing | **$0.14/LINEAR FOOT** | **$0.21/LINEAR FOOT** | Lawn maintenance area = (top width + 10 ft) x length. Mow eight times per year |
| General lawn care | 9.00 / 1000 $ft^2$ / year | **$0.18/LINEAR FOOT** | **$0.28/LINEAR FOOT** | Lawn maintenance area = (top width + 10 ft)( x length |
| Swale debris and litter removal | $0.10 / linear foot / year | **$0.10/LINEAR FOOT** | **$0.10/LINEAR FOOT** | - |
| Grass reseeding with mulch and fertilizer | $0.30 / $yd^2$ | **$0.01/LINEAR FOOT** | **$0.01/LINEAR FOOT** | Area revegetated equals 1% of lawn maintenance area per year |
| Program administration and swale inspection | $0.15 / linear foot / year plus $25 / inspection | **$0.15/LINEAR FOOT** | **$0.15/LINEAR FOOT** | Inspect 4 times per year |
| Total | - | **$0.58/LINEAR FOOT** | **$0.75/LINEAR FOOT** | - |

*E1-2.4.9.3 Performance*

Uncertainties about performance include:

- Best designs for optimum pollutant removal capability
- Relationship of removal rates to age
- Benefits of check dams or steps in preventing downstream migration of pollutants
- Effects of different grass types and height on pollutant removal capability

*E1-2.4.9.4 Construction Guidelines and Cost*

**The wide range of cost data reported in**

Table E1-15 indicates that much more information is needed in this area. Do the differences in cost reflect different regulatory requirements, differences in construction techniques, or perhaps site suitability? Identifying the primary elements that drive construction costs upward is a high priority and could help reduce the initial cost associated with this widely used BMP / SUDS.

*E1-2.4.9.5 Maintenance Guidelines and Cost*

Maintenance costs for the Orange County area (Table E1-17) are well documented and no additional work is recommended in this area.

E1-2.4.10 References

Backstrom, M. , 2001. "Particulate Trapping In Grass Swales," *Novatech 2001-Innovative Technologies In Urban Drainage Conference Proceedings, 25-27 June 2001, Lyon, France*. GRAIE, Villeurbanne, Lyon, France. ISBN 2 9509337-4-2. pp 391-398.

Balades, J.D. and Petitnicolas, F., 2001. "Strategies of Wet Weather Pollution Control at Source. A Technical and Financial Approach," *Novatech 2001-Innovative Technologies In Urban Drainage Conference Proceedings*, 25-27 June 2001, Lyon, France. GRAIE, Villeurbanne, Lyon, France. ISBN 2 9509337-4-2. pp 367-373.

Baptista, M., Barraud, S., Alfakih, E., 2001. "Data Analysis for the Elaboration of Technical and Economic Indicators of Alternative Storm Drainage Systems," *Novatech 2001-Innovative*

*Technologies In Urban Drainage Conference Proceedings,* 25-27 June 2001, Lyon, France. GRAIE, Villeurbanne, Lyon, France. ISBN 2 9509337-4-2. pp 63.70.

Barrett, Michael E., Walsh, Patrick M., Malina, Joseph F., Jr., Charbeneau, Randall J., 1998. “Performance Of Vegetative Controls For Treating Highway Runoff,” *ASCE Journal of Environmental Engineering,* Vol. 124, No. 11, pp. 1121-1128.

Brown, W., and T. Schueler, 1997. *The Economics of Stormwater BMPs in the Mid-Atlantic Region.* Prepared for the Chesapeake Research Consortium, Edgewater, MD, by the Center for Watershed Protection, Ellicott City, MD.

Center for Watershed Protection (CWP), 1996. *Design of Stormwater Filtering Systems*. Prepared for the Chesapeake Research Consortium, Solomons, MD, and USEPA Region V, Chicago, IL, by the Center for Watershed Protection, Ellicott City, MD.

Colwell, Shanti R., Horner, Richard R., and Booth, Derek B., 2000. *Characterization of Performance Predictors and Evaluation of Mowing Practices in Biofiltration Swales*. Report to King County Land And Water Resources Division and others by Center for Urban Water Resources Management, Department of Civil and Environmental Engineering, University of Washington, Seattle, WA.

Construction Industry Research and Information Association, 2000. *Sustainable Urban Drainage Systems- Design Manual for Scotland and Northern Ireland*. CIRA C521, ISBN 0 86017 521 9. *Sustainable Urban Drainage Systems – Design Manual for England and Wales.* CIRIA C522, ISBN 0 86017 522 7.

Dorman, M.E., J. Hartigan, R.F. Steg, and T. Quasebarth, 1989. *Retention, Detention and Overland Flow for Pollutant Removal From Highway Stormwater Runoff. Vol. 1.* FHWA/RD 89/202. Federal Highway Administration, Washington, DC.

Goldberg, 1993. *Dayton Avenue Swale Biofiltration Study*. Seattle Engineering Department, Seattle, WA.

Harper, H. Yousef, Y., and Wanielista, M., 1984. “Efficiency of Roadside Swales in Removing Heavy Metals from Highway Associated Nonpoint Source Runoff,” AWRA Options for Reaching Water Quality Goals.

Harper, H., 1988. *Effects of Stormwater Management Systems on Groundwater Quality*. Prepared for Florida Department of Environmental Regulation, Tallahassee, FL, by Environmental Research and Design, Inc., Orlando, FL.

Honeyman and Baird, 2001. Newhall Valley Water Management Strategy. First National Conference on Sustainable Drainage, Coventry, UK.

Horner, R.R., Skupien, J.J., Livingston, E.H. and Shaver, H.E., 1994. *Fundamentals of Urban Runoff Management: Technical and Institutional Issues*. Terrene Institute, Washington, DC.

Kercher, W.C., J.C. Landon, and R. Massarelli, 1983. "Grassy Swales Prove Cost-Effective For Water Pollution Control," *Public Works*, 16: 53–55.

Koon, J., 1995. *Evaluation of Water Quality Ponds and Swales in the Issaquah/East Lake Sammamish Basins*. King County Surface Water Management, Seattle, WA, and Washington Department of Ecology, Olympia, WA.

Lawrence, A.I., Marsalek J., Ellis, J.B., Urbonas B., 1996. "Stormwater Detention and BMPs," *J. Hydr. Res*. vol. 34, no. 6, pp 799-813.

Macdonald K., 2002. PhD *Thesis in preparation*. University of Abertay, Dundee, Scotland

MacDonald and Jefferies, 2002. Performance of Two Swales. Scottish Hydraulics Study Group, 14th Annual Seminar.

N.C. Department of Environment and Natural Resources, 1999. *Stormwater Best Management Practices*. Division of Water Quality Water Quality Section. Department of Environment and Natural Resources, NC.

Oakland, P.H., 1983. "An Evaluation of Stormwater Pollutant Removal Through Grassed Swale Treatment," in *Proceedings of the International Symposium of Urban Hydrology, Hydraulics and Sediment Control, Lexington, KY*, pp. 173–182.

Occoquan Watershed Monitoring Laboratory, 1983. *Final Report: Metropolitan Washington Urban Runoff Project*. Prepared for the Metropolitan Washington Council of Governments, Washington, DC, by the Occoquan Watershed Monitoring Laboratory, Manassas, VA.

Pitt, R., and J. McLean, 1986. *Toronto Area Watershed Management Strategy Study: Humber River Pilot Watershed Project.* Ontario Ministry of Environment, Toronto, ON.

Roesner, L.A., Howard, R.M., Mack, B.W. and Ramdett C.A., 1997. "Integrating Stormwater Management into Urban Planning in Orlando, Florida," *Sustaining Urban Water Resources in the 21st Century'*, Engineering Foundation Conference, Malmo, Sweden, 8-12 September 1997.

Schueler, T. 1997. "Comparative Pollutant Removal Capability of Urban BMPs: A Reanalysis," *Watershed Protection Techniques* 2(2):379–383.

Seattle Metro and Washington Department of Ecology. 1992. *Biofiltration Swale Performance: Recommendations and Design Considerations*. Publication No. 657. Water Pollution Control Department, Seattle, WA.

Scottish Environment Protection Agency and Environment Agency, 1999. *Sustainable Urban Drainage Systems- an Introduction* Joint Agency Publication, ISBN 1-901322-12-8.

Southeastern Wisconsin Regional Planning Commission (SWRPC). 1991. *Costs of Urban Nonpoint Source Water Pollution Control Measures.* Technical report no. 31. Southeastern Wisconsin Regional Planning Commission, Waukesha, WI.

United States Environmental Protection Agency, 1999. *Stormwater Technology Fact Sheet*, Vegetated Swales. 832-F-99-006. Office of Water Washington, DC.

Wang, T., D. Spyridakis, B. Mar, and R. Horner, 1981. *Transport, Deposition and Control of Heavy Metals in Highway Runoff*. FHWA-WA-RD-39-10. University of Washington, Department of Civil Engineering, Seattle, WA.

Wanielista, M, Gennaro, R., Bell, J., Johnson, W., 1978. *Shallow Roadside Ditches for Stormwater Purification*, Florida Department of Transportation Report FL-ER-3-78.

Washington State Department of Transportation, 1995, *Highway Runoff Manual*, Washington State Department of Transportation, Olympia, WA.

Watershed Protection Techniques, 1996. “Ditches or Biological Filters? Classifying the Pollutant Removal Performance of Open Cannels. Technical Notes-Urban Best Management Practices,” *Watershed Protection Techniques*, vol. 2, no. 2. Spring 1996.

Welborn, C., and J. Veenhuis. 1987. *Effects of Runoff Controls on the Quantity and Quality of Urban Runoff in Two Locations in Austin, TX.* USGS Water Resources Investigations Report No. 87-4004. U.S. Geological Survey, Reston, VA.

Wigington, P.J., Jr., Randall, C.W., and Grizzard, T.J., 1983. “Accumulation of Selected Trace Metals in Soils of Urban Runoff Detention Basins,” *Water Resources Bulletin*, vol. 19, no. 5, pp 709-718.

Yousef, Y., Hitved-Jacobsen, T., Wanielista, M., Harper, H., 1987. “Removal of Contaminants in Highway Runoff Flowing Through Swales,” *The Science of the Total Environment*, vol. 59, pp 391-399.

Yousef, Y., Wanielista, M., Harper, H., 1985. “Removal of Highway Contaminants by Roadside Swales,” *Transportation Research Record*, vol. 1017, pp 62-68.

Yousef, Y., M. Wanielista, H. Harper, D. Pearce, and R. Tolbert. 1985. *Best Management Practices: Removal of Highway Contaminants By Roadside Swales*. University of Central Florida and Florida Department of Transportation, Orlando, FL.

Yu, S., S. Barnes, and V. Gerde. 1993. *Testing of Best Management Practices for Controlling Highway Runoff*. FHWA/VA-93-R16. Virginia Transportation Research Council, Charlottesville, VA.

**Information Resources**

Maryland Department of the Environment (MDE), 2000. *Maryland Stormwater Design Manual.* [www.mde.state.md.us/environment/wma/stormwatermanual]. Accessed May 22, 2001.

Reeves, E., 1994. "Performance and Condition of Biofilters in the Pacific Northwest," *Watershed Protection Techniques* 1(3):117–119.

Seattle Metro and Washington Department of Ecology, 1992. *Biofiltration Swale Performance.* Recommendations and Design Considerations. Publication No. 657. Seattle Metro and Washington Department of Ecology, Olympia, WA.

USEPA, 1993. *Guidance Specifying Management Measures for Sources of Nonpoint Pollution in Coastal Waters*. EPA-840-B-92-002. U.S. Environmental Protection Agency, Office of Water. Washington, DC.

Watershed Management Institute (WMI). 1997, *Operation, Maintenance, and Management of Stormwater Management Systems*. Prepared for U.S. Environmental Protection Agency, Office of Water. Washington, DC, by the Watershed Management Institute, Ingleside, MD.

## E1-2.5 Vegetated Buffer Strips

### E1-2.5.1 Description

Grassed buffer strips (vegetated filter strips, filter strips, and grassed filters) are vegetated surfaces that are designed to treat sheet flow from adjacent surfaces. Filter strips function by slowing runoff velocities and allowing sediment and other pollutants to settle, and by providing some infiltration into underlying soils. Buffer strips were originally used as an agricultural treatment practice, and have more recently evolved into an urban practice. With proper design and maintenance, buffer strips can provide relatively high pollutant removal. These strips tend to break up large contiguous impervious areas, which can create more opportunities for retention and treatment of runoff. In addition, the public views them as landscaped amenities and not as stormwater infrastructure; consequently, there is little resistance to their use.

Buffer strips can be divided into two basic types of vegetative stormwater quality management practices: (1) natural buffers; and (2) engineered biofilters. Both systems encourage diffuse, shallow, low velocity flow across a vegetated surface and both can provide effective stormwater quality enhancement if flow and vegetative conditions are correct. Natural vegetated buffers are typically left undisturbed and are used in areas with relatively low-density development as a passive, low maintenance means of protecting nearby receiving waters from marginally increased pollutant loads.

Engineered biofilters are specifically designed and constructed to maximize the water quality benefits of vegetative filtration practices, particularly in areas where adequate buffers do not exist naturally or can not be preserved. Engineered systems feature mechanically graded surfaces with vegetation selection targeted to physical site conditions, water quality performance, and aesthetic appeal. Figure E1-18 presents a schematic representation of an engineered biofilter, while Figure E1-19 shows a photograph of an actual installation.

### E1-2.5.2 Advantages

- Buffers require minimal maintenance activity (generally just erosion prevention and mowing).
- If properly designed, vegetated, and operated, buffer strips can provide reliable water quality benefits in conjunction with high aesthetic appeal.
- Flow characteristics and vegetation type and density can be closely controlled to maximize BMP effectiveness.
- Roadside shoulders act as effective buffer strips when slope and length meet criteria described in the subsequent sections.
- Buffers have a multi-purpose versatility for aesthetic landscape, informal play areas, etc.

**Figure E1-18 Engineered Filter Strip Schematic**

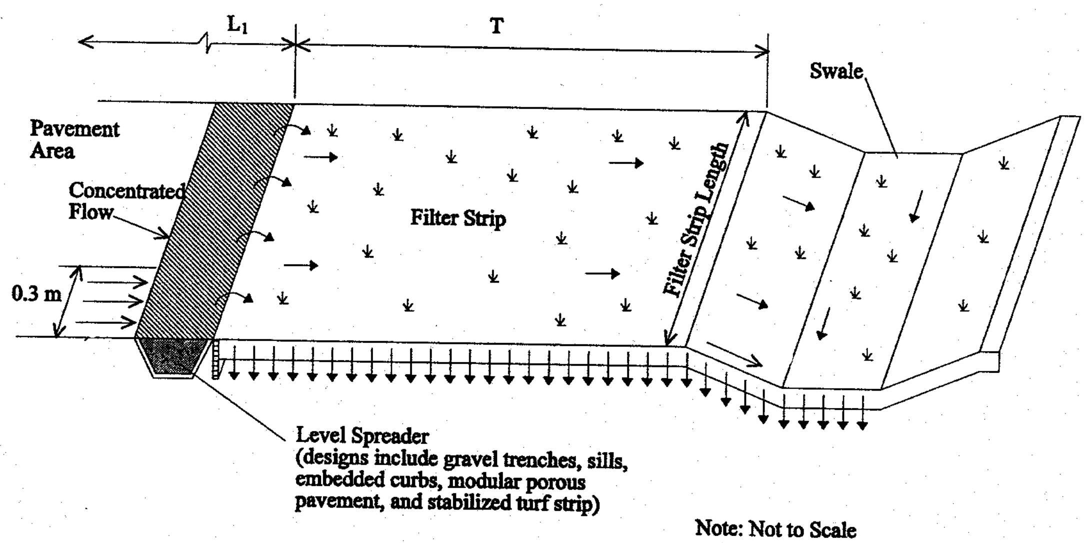



**Figure E1-19 Filter Strip, Hopwood Services, UK**

### E1-2.5.3 Limitations

Buffer strips have some limitations, including the following:

- Buffer strips cannot treat a very large drainage area.
- They are impractical in areas with steep topography, or wet or poorly drained soils.
- They are not effective and may even erode when flow volumes and/or velocities are high.
- Sufficient land may not be available for suitable buffer strip designs to be incorporated.
- Many local municipalities/councils in the US and UK may require curb and gutter systems in residential areas.
- They are impractical in areas with erosive soils or where a dense vegetative cover is difficult to maintain.
- Infiltration through the filter strip may carry pollutants into local groundwater.
- Filter strips require sheet flow. Piping surface water to the strip is not recommended as this may cause erosion, and minimize filtration opportunities.

### E1-2.5.4 Siting Criteria

The use of natural or engineered biofilters is limited to gently sloping areas where the vegetative cover is robust and diffuse, and where shallow flow characteristics are possible. The practical water quality benefits of each practice can be effectively eliminated with the occurrence of significant erosion or when flow concentration occurs across the vegetated surface. Slopes should not exceed 10 percent or be less than 1 percent. The vegetative surface should extend across the full width of the area being drained. The upstream boundary of the filter should be located contiguous to the developed area. Use of a level spreading device (vegetated berm, sawtooth concrete border, rock trench, etc) to facilitate overland sheet flow is not normally recommended because of maintenance considerations and the potential for standing water.

Filter strips are applicable in most regions, but are restricted in some situations because they consume a large amount of space relative to other practices. They are estimated to require 5 to 10 percent of the catchment area for implementation. Filter strips are best suited to treating runoff from roads and highways, roof downspouts, very small parking lots, and pervious surfaces. They are also ideal components of the "outer zone" of a stream buffer, or as pretreatment to a structural practice. In arid areas, however, the cost of irrigating the grass on the practice will most likely outweigh its water quality benefits, although aesthetic considerations may be sufficient to overcome this constraint. Ultra-urban areas are densely developed urban areas in which little pervious surface exists. Filter strips are impractical in these areas because of space constraints. Filter strips also are generally a poor retrofit option because they consume a relatively large amount of space and cannot treat large drainage areas.

In many cases, vegetated areas are provided in many site designs where their potential for runoff treatment as an additional amenity is actively ignored. For instance vegetation is often included in parking lots, but in raised areas surrounded by curbs and isolated from stormwater.

Some cold water species, such as trout, are sensitive to changes in temperature. While some treatment practices, such as wet ponds, can warm stormwater substantially, filter strips do not warm pond water on the surface for long periods of time and are not expected to increase stormwater temperatures. Thus, these practices are good for protection of cold-water streams.

### E1-2.5.5 Design Guidelines

Filter strips appear to be a minimal design practice because they are basically no more than a grassed slope.

A major question that remains unresolved is how large the drainage area to a strip can be. Research has conclusively demonstrated that these are effective on roadside shoulders, where the contributing area is about twice the buffer area. They have also been installed on the perimeter of large parking lots where they performed fairly effectively; however much lower slopes may be needed to provide adequate water quality treatment.

For engineered vegetative strips, the facility surface should be graded flat prior to placement of vegetation. Initial establishment of vegetation requires attentive care including appropriate watering, fertilization and prevention of excessive flow across the facility until vegetation completely covers the area and is well established. Use of a permanent irrigation system may help provide maximal water quality performance.

In cold climates, filter strips provide a convenient area for snow storage and treatment. If used for this purpose, vegetation in the filter strip should be salt-tolerant, (e.g., creeping bentgrass), and a maintenance schedule should include the removal of sand built up at the bottom of the slope. In arid or semi-arid climates, designers should specify drought-tolerant grasses (e.g., buffalo grass) to minimize irrigation requirements.

### E1-2.5.6 Performance

Vegetated buffer strips tend to provide somewhat better treatment of stormwater runoff than swales and have fewer tendencies for channelization or erosion. Table E1-20 documents the pollutant removal observed in a recent study by Caltrans (2002) based on three sites in southern California. The column labeled "Significance" is the probability that the mean influent and effluent EMCs are not significantly different based on an ANOVA.

The removal of sediment and dissolved metals was comparable to that observed in much more complex controls. Reduction in nitrogen was not significant and all of the sites exported phosphorus for the entire study period. This may have been the result of using salt grass, a warm weather species that is dormant during the wet season.

**Table E1-19 Design Guidelines for Vegetated Buffers**

| Design Criteria | Current guidance / recommendations |
|---|---|
| Planting | • The filter area should be densely vegetated with a mix of erosion-resistant plant species that effectively bind the soil. Native or adapted grasses, shrubs, and trees are preferred because they generally require less fertilizer and are more drought resistant than exotic plants. |
| Slope | • In general the slope of the strip should not exceed 10%. The gentler the slope, the greater its effectiveness for pollutant removal.<br>• Both the top and toe of the slope should be as flat as possible to encourage sheet flow and prevent erosion. |
| Filter strip length | • The strip should be at least 5 -7 m (16-23 ft) long to provide water quality treatment. |
| Flow Velocities | • Runoff flow velocities should not exceed about 0.3 m/s (1 ft/s) across the vegetated surface to encourage settlement during regular storms.<br>• Flow velocities for larger floods should be below 1.5 m/s (5 ft/sec) to prevent erosion. |
| Inlet Infrastructure | • The top of the strip should be installed 50 to 125 mm (2 to 5 in) below the adjacent pavement, so that vegetation and sediment accumulation at the edge of the strip does not prevent runoff from entering. |
| Subsoil | • Topsoil should be lightly compacted to a depth of 100 mm (4 in). |
| Water Table | • Geotechnical tests must be performed to determine the location of the water table. If the water table is within 0.6 m (2 ft) of the planned swale bottom, a dry swale is not feasible. ([1])<br>• If there is a risk of a swale conveying potential polluted water, and the groundwater requires protection, an impermeable liner should be incorporated in the design (CIRIA, 2001) |
| Groundwater levels | • Filter strips should be separated from the ground water by between 0.6 to 1.2 m (2 and 4 ft) to prevent contamination and to ensure that the filter strip does not remain wet between storms. |

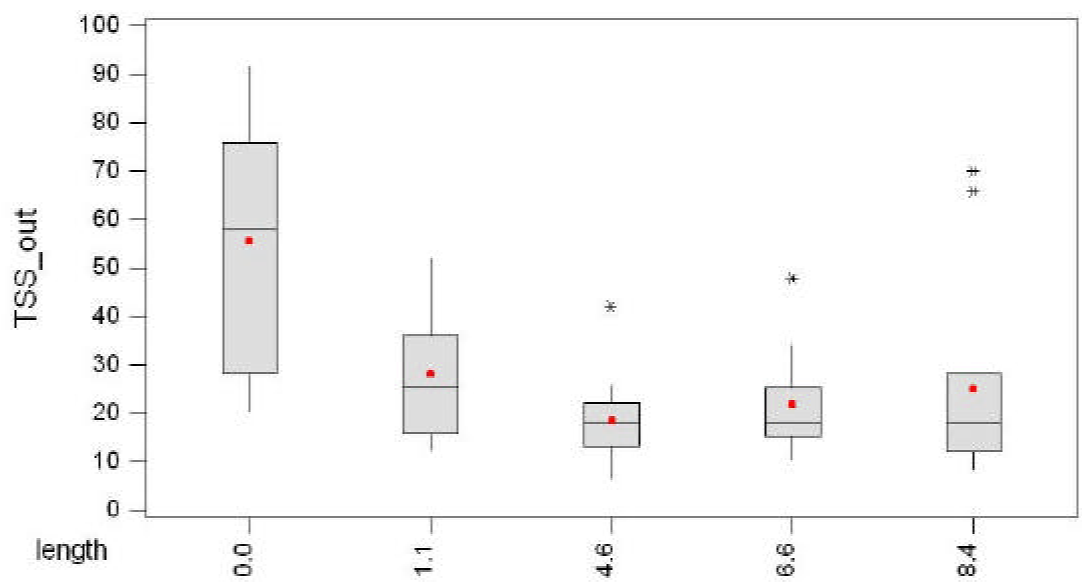
**Table E1-20 Pollutant Reduction in a Vegetated Buffer Strip**

| Constituent | Mean EMC | | Removal,% | Significance, P |
|---|---|---|---|---|
| | Influent mg/L | Effluent mg/L | | |
| TSS | 119 | 31 | 74 | <0.000 |
| $NO_3$-N | 0.67 | 0.58 | 13 | 0.367 |
| TKN-N | 2.50 | 2.10 | 16 | 0.542 |
| Total N[a] | 3.22 | 2.83 | 12 | - |
| Dissolved P | 0.15 | 0.46 | -206 | 0.047 |
| Total P | 0.42 | 0.62 | -52 | 0.035 |
| Total Cu | 0.058 | 0.009 | 84 | <0.000 |
| Total Pb | 0.046 | 0.006 | 88 | <0.000 |
| Total Zn | 0.245 | 0.055 | 78 | <0.000 |
| Dissolved Cu | 0.029 | 0.007 | 77 | 0.004 |
| Dissolved Pb | 0.004 | 0.002 | 66 | 0.006 |
| Dissolved Zn | 0.099 | 0.035 | 65 | <0.000 |

Another Caltrans study (unpublished) of vegetated highway shoulders as buffer strips also found substantial reductions, often within a very short distance of the edge of pavement. Figure E1-20 presents the concentrations of TSS in highway runoff after traveling various distances (shown in meters) through a vegetated filter strip with a slope of about 10 percent. One can see that the TSS reaches an irreducible minimum concentration of about 20 mg/L within 5 m (16 ft) of the pavement edge.

Filter strips also exhibit good removal of litter and other floatables because the water depth in these systems is well below the vegetation height and consequently these materials are not easily transported through them. Unfortunately little attenuation of peak runoff rates and volumes (particularly for larger events) is normally observed, depending on the soil properties. Therefore it may be prudent to follow the strips with another practice than can reduce flooding and channel erosion downstream.

**Figure E1-20 TSS Concentration in Buffer Strip Discharge**




*Note: Length is in meters.

E1-2.5.7 Maintenance

Filter strips require mainly vegetation management; therefore little special training is needed for maintenance crews. Typical maintenance activities and frequencies are listed in Table E1-21.

E1-2.5.8 Cost

*E1-2.5.8.1 Construction Costs*

Little data are available on the actual construction costs of filter strips. One rough estimate can be the cost of seed or sod, which is approximately \$3.20 / $m^3$ (\$0.30 / $ft^2$) for seed or \$7.50 / $m^3$ (\$0.70 / $ft^2$) for sod. This amounts to between \$13,000 and \$30,000 per acre of filter strip. This cost is relatively high compared with other treatment practices. However, the grassed area used as a filter strip may have been seeded or sodded even if it were not used for treatment. In these cases, the only additional costs are the design.

**Table E1-21 Typical Maintenance Activities for Filter Strips**

| Activity | Schedule |
| --- | --- |
| • Inspect strips at least twice annually for erosion or damage to vegetation, preferably at the end of the wet season to schedule summer maintenance and before major fall runoff to be sure the strip is ready for winter. However, additional inspection after periods of heavy runoff is most desirable to check for rills, gullies or water logging. | Biannually |
| • The strip should be checked regularly for debris and litter, and areas of sediment accumulation.<br><br>• Trash tends to accumulate in strip areas, particularly along highways. The need for litter removal should be determined through periodic inspection, but litter should always be removed prior to mowing. | As needed |
| • Recent research on biofiltration swales, but likely applicable to strips (Colwell et al., 2000) indicates that grass height and mowing frequency have little impact on pollutant removal; consequently, mowing may only be necessary once or twice a year for safety and aesthetics or to suppress weeds and woody vegetation. | Annually |
| • Sweeping and erosion control of the upstream catchment should be undertaken to reduce opportunities for silt to enter the system. | Monthly |

The true cost of filter strips is the land they consume, which is higher than for any other treatment practice. In some situations this land is available as wasted space beyond back yards or adjacent to roadsides, but this practice is cost-prohibitive when land prices are high and land could be used for other purposes.

*E1-2.5.8.2 Maintenance Costs*

Maintenance of vegetated buffer strips consists mainly of vegetation management (mowing, irrigation if needed, weeding) and litter removal. Consequently the costs are quite variable depending on the frequency of these activities and the local labor rate.

Typical maintenance costs are about $350 / acre / year (adapted from SWRPC, 1991). This cost is relatively inexpensive and, again, might overlap with regular landscape maintenance costs.

## E1-2.5.9 Research Needs

### *E1-2.5.9.1 Design Guidelines*

- Size of drainage area.

### *E1-2.5.9.2 Performance*

- Effects of mowing on pollutant removal capability of grassed filter strips.
- Effects of different grass types on pollutant removal capability.
- Whether removal rates decline with age.

### *E1-2.5.9.3 Maintenance*

- Optimum frequency to retain optimum grass lengths.

### *E1-2.5.9.4 Cost*

- Construction out-turn (as-built) costs required.

## E1-2.5.10 References

Caltrans, 2002. *BMP Retrofit Pilot Program Proposed Final Report*, CTSW-RT-01-050, California Department of Transportation, Sacramento, CA (in press).

Center for Watershed Protection (CWP), 1996. *Design of Stormwater Filtering Systems.* Prepared for Chesapeake Research Consortium, Solomons, MD, and EPA Region V, Chicago, IL.

Desbonette, A., P. Pogue, V. Lee, and N. Wolff, 1994. *Vegetated Buffers in the Coastal Zone: A Summary Review and Bibliography*. Coastal Resources Center. University of Rhode Island, Kingston, RI.

Magette, W., R. Brinsfield, R. Palmer and J. Wood, 1989. Nutrient and Sediment Removal by Vegetated Filter Strips. *Transactions of the American Society of Agricultural Engineers* 32(2): 663–667.

Southeastern Wisconsin Regional Planning Commission (SWRPC), 1991. *Costs of Urban Nonpoint Source Water Pollution Control Measures.* Technical report no. 31. Southeastern Wisconsin Regional Planning Commission, Waukesha, WI.

Yu, S., S. Barnes and V. Gerde, 1993. *Testing of Best Management Practices for Controlling Highway Runoff*. FHWA/VA 93-R16. Virginia Transportation Research Council, Charlottesville, VA.

**Information Resources**

Center for Watershed Protection (CWP), 1997. *Stormwater BMP Design Supplement for Cold Climates*. Prepared for U.S. Environmental Protection Agency Office of Wetlands, Oceans and Watersheds. Washington, DC.

Maryland Department of the Environment (MDE), 2000. *Maryland Stormwater Design Manual.* [http://www.mde.state.md.us/environment/wma/stormwatermanual]. Accessed May 22, 2001.

## E1-2.6 Infiltration Basins

### E1-2.6.1 Description

**An infiltration basin (Figure E1-21, photograph and**

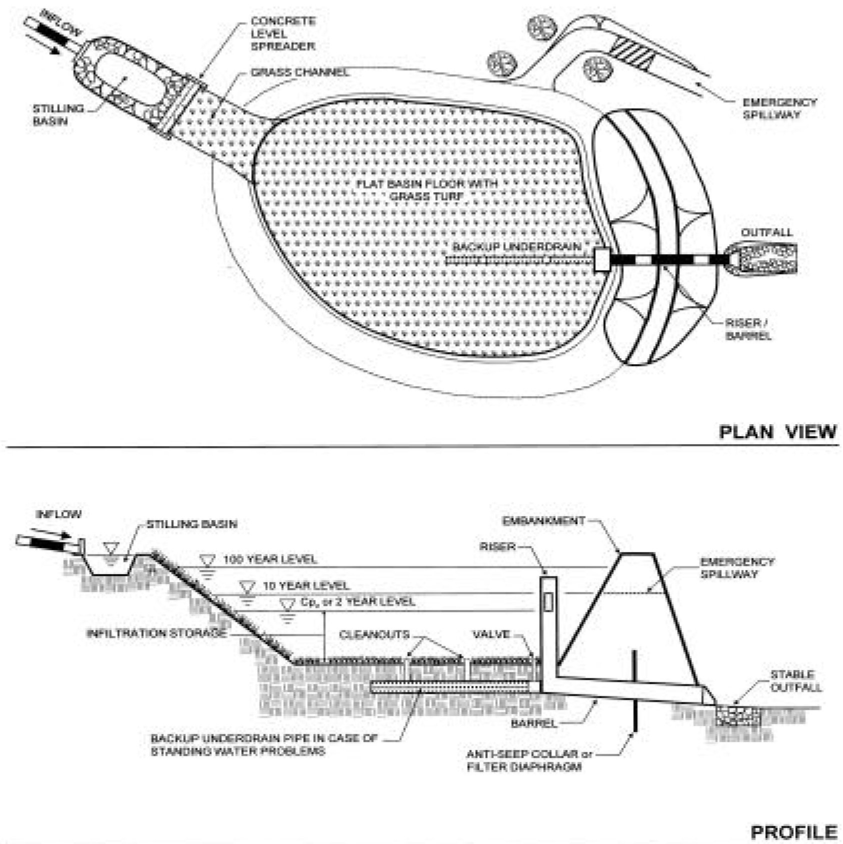
Figure E1-22 schematic diagram) is a shallow impoundment which is designed to infiltrate stormwater into the ground. This practice has high pollutant removal efficiency and can also help recharge the groundwater, thus restoring low flows to stream systems. Infiltration basins should only be used as part of a "treatment train," where soluble organic substances, oils, and course sediment are removed by other management practices prior to stormwater entering the infiltration basin. This practice should not be used in industrial parks, high density or heavy industrial areas, chemical or pesticide storage areas, or fueling stations where there may be a risk to groundwater contamination.

Infiltration basins can be challenging to apply on many sites because of soil characteristics. In addition, some studies have shown relatively high failure rates compared with other management practices as a result of surface clogging and reduction in infiltration potential.

Infiltration Basins are more commonly used in North America and most of the information given in this report is based on US experiences. No British-based performance data are available but a number of studies have been undertaken in France (CIRIA, 1999). The main difficulty associated with monitoring these systems is that almost all water infiltrates through the basin and quality monitoring would involve assessing the change in soil water/groundwater quality, and there is little data in this area.

**Figure E1-21 Infiltration Basin I-605/SR-91, Los Angeles County**

**Figure E1-22 Schematic of an Infiltration Basin**




E1-2.6.2 <u>Advantages</u>

- The principal benefit of infiltration basins is the approximation of pre-development hydrology during which a significant portion of the average annual rainfall runoff is infiltrated and evaporated rather than flushed directly to receiving waters.
- If the infiltration basin volume is adequately sized, the systems can be very useful for providing control of channel forming (erosion) and high frequency (generally less than the 2-year) flood events.
- Although infiltration basins are deemed suitable for treating relatively small catchment areas ranging from 2 to 6 ha (5 to 15 acres) (Schueler, 1987), experience at Venissieux, France, and Fresno, California, shows that they can serve much larger areas.

E1-2.6.3 <u>Limitations</u>

- Infiltration basins have potentially high failure rates. The main reasons being improper siting, design and lack of maintenance. Proper pre-treatment needs to be incorporated in the design and is essential for the longevity of this device.

- Soil conditions are the most critical factor for the site selection. In the US, a minimum infiltration rate of 0.5 inch / hour is required and sites with Soil Type C and D (soil types with high clay content) are excluded. A minimum distance to the groundwater table is required to ensure sufficient hydraulic gradient to meet the specified drain time.

- Infiltration basins are not appropriate for treating flows with significant loads of sediment and other pollutants due to the potential for clogging. They are also not appropriate for industrial sites or locations where spills may occur.

- Frequent inspections and maintenance have to be undertaken and an aggressive maintenance schedule must be adopted.

- Infiltration basins are not suitable on fill sites or steep slopes, or where there is a risk of groundwater contamination due to the soil structure. They should not be employed adjacent to drinking water wells for this reason.

- The proximity of infiltration basins to other structures, including foundation, must be taken into account during the design phase due to the risks of changes in soil structure as a result of the infiltration processes. Current UK Building Regulations advise that infiltration systems should not be constructed within 5 m (16 ft) of roads or building foundations.

### E1-2.6.4 Siting Criteria

The key element in siting infiltration basins is identifying sites with appropriate soil and hydrogeologic properties, which is critical for long term performance. In one study conducted in Prince George's County, Maryland (Galli, 1992), all of the infiltration basins investigated clogged within 2 years. This trend may not be the same in soils with high infiltration rates, however. A study of 23 infiltration basins in the Pacific Northwest showed better long-term performance in an area with highly permeable soils (Hilding, 1996). In this study, few of the infiltration basins had failed after 10 years. Infiltration basins require about 0.5 to 2 percent of the catchment area for implementation.

The following guidelines for identifying appropriate soil and subsurface conditions should be followed.

- Determine soil type from mapping, soil survey tables and onsite investigation to estimate permeability.

- Eliminate sites with high ground water levels.

- Locate away from buildings, slopes and highway pavement and wells and bridge structures.

- Do not consider sites constructed of fill, having a base flow, or with steep slope.

- Ensure that adequate head is available to operate flow splitter structures, such as weirs, designed to divert the water quality volume to the infiltration basin.

Many guidelines recommend against the use of infiltration basins in catchments prone to high pollutant loads or spills of toxic materials because of potential groundwater contamination. However, the USEPA (1983) and Nightingale (1975; 1987a, b, c; 1989) found no evidence of transport of pollutants for any distance in the subsurface. For instance, a report by Pitt et al. (1994) highlighted the potential for groundwater contamination from intentional and unintentional stormwater infiltration. That report recommends that infiltration facilities not be sited in areas where high concentrations are present or where there is a potential for spills of toxic material. Conversely, Schroeder (1995) reported that there was no evidence of groundwater impacts from an infiltration basin serving a large industrial catchment in Fresno, CA.

### E1-2.6.5 Design Guidelines

**Table E1-22 Design Guidelines for Infiltration Basins**

| Design Criteria | Current Guidance/Recommendations |
|---|---|
| Soil characteristics | • Infiltration basins require an extensive geotechnical exploration to determine the subsurface profile and the hydraulic conductivity of the *in situ* soils.<br>• In the US pre-construction soil infiltration rate in the soil should be between 13 mm/hr (0.51 in/hr) and 100 mm/hr (4 in/hr). |
| Groundwater levels | • Historical well records and geotechnical investigations must also be evaluated to establish potential ground water levels.<br>• The invert of infiltration basins should be located at least 1.2 m (4 ft) above the seasonally high groundwater elevation. |
| Inlet Infrastructure | • Inflow to the basin may be by sheet flow down the sides, riprap/concrete chute for concentrated surface flows from paved areas, or a drain discharging from the basin side with suitable scour protection and headworks (CIRIA, 2001).<br>• Include energy dissipation in the inlet design for the basins to minimize scour potential.<br>• Avoid designs that include a permanent pool to reduce opportunity for standing water and associated vector problems. |
| Pretreatment | • Provide pretreatment if sediment loading is a maintenance concern for the basin. |
| Flood Control/ Treatment Volumes | • The required capture volume is determined by local regulations, i.e., in CA capture 85% of the annual rainfall depth.<br>• Infiltration basins shall be designed to capture, store and treat (infiltrate) the Water Quality Volume.<br>• Basin sized so that the entire water quality volume is infiltrated within 48 hours.<br>• Infiltration/retention times longer than 48 hours should not be considered to minimize the potential for mosquito breeding.<br>• After a storm with a short recurrence interval (< 2 years), the infiltration of water should be sufficient to drain half the stored runoff within 24 hours. In extreme storms it will take longer to empty an infiltration basin (CIRIA, 2001). |

| Design Criteria | Current Guidance/Recommendations |
|---|---|
| Basin Invert Area and Shape | • Basin invert area should be determined by the equation: $$A = \frac{WQV}{kt}$$ Where: A = Basin invert area ($m^2$) WQV = water quality volume ($m^3$) k = 0.5 times the lowest field-measured hydraulic conductivity (m / hr) t = drawdown time (48 hr) <br>• Infiltration basins should have gently sloping sides and a flat base area. <br>• Side slope should be 1 in 4 or shallower (this is considered to be the steepest slope which can easily be negotiated by someone trying to exit the basin and that can be mowed with a ride-on mower) (CIRIA, 2001). <br>• There should be a smooth transition between the base and sides of the infiltration basin. <br>• Curving, irregular plan forms are preferred from an amenity and visual standpoint. <br>• Depths of construction generally range from 0.5 – 3.0 m (0.15 – 0.9 ft). A freeboard should be maintained above the maximum water level, but this should be based on the calculated risks of overtopping. |
| Vertical Piping | • In the US the use of vertical piping, either for distribution or infiltration enhancement shall not be allowed to avoid device classification as a Class V injection well per 40 Code of Fed. Reg. 146.5(e) (4). |
| Catchment Topography | • Land Slope: Infiltration basins can be located on slopes of up to 15 percent. |
| Outlet Infrastructure | • Infiltration basins shall have an overflow outlet or bypass to limit the risk of overtopping the device. Alternatively, detention storage can be built on top of the infiltration basin. |
| Vegetation | • The infiltration basin sides and bottom should be stabilized with vegetation or non-vegetative measures to minimize erosion and controls dust. Using vegetation at bottom can improve the infiltration characteristics. |
| Maintenance needs | • Dedicated access to the basin bottom should be provided for maintenance vehicles. <br>• Basins may be lined with a layer of filter material such as coarse sand to prevent the buildup of impervious deposits on the natural soil surface. |
| Safety Measures | • Barrier planting (or a vehicle barrier if next to a road) should be considered where any significant pool of water is likely to remain following a storm. <br>• At infiltration basins used for a secondary purpose, notices should be erected at the site. In conjunction with education of the local residents, such measures should emphasize that the basin is primarily a drainage device and inform people how to use it safely. |

### E1-2.6.6 Performance

As water migrates through porous soil and rock, pollutant attenuation mechanisms include precipitation, sorption, physical filtration, and bacterial degradation. If functioning properly, this approach is presumed to have high removal efficiencies for particulate pollutants and moderate removal of soluble pollutants. Actual pollutant removal in the subsurface would be expected to vary depending upon site-specific soil types.

This technology eliminates discharge to surface waters except for the very largest storms; consequently, complete removal of all stormwater constituents can be assumed.

Performance (relative to surface water discharge)

Sediment: 100%
Total Nitrogen: 100%
Total Phosphorus: 100%
Dissolved Zinc: 100%
Toxic Materials: 100%
Litter: 100%
Oxygen Demanding Substances: 100%
Oil & Grease: 100%
Bacteria: 100%

Qualitative performance information has been gleaned from some parts of the world, as indicated below:

**California**

Infiltration basins have a long history of use in California, especially in the Central Valley. Basins located in Fresno were among those initially evaluated in the National Urban Runoff Program, and were found to be effective at reducing the volume of runoff, while posing little threat to groundwater quality (EPA, 1983).

Proper siting of these devices is crucial as underscored by the experience of Caltrans in siting two basins in Southern California. The basin with marginal separation from groundwater failed immediately and could never be rehabilitated.

**Bordeaux, France**

An infiltration basin, holding 80,000 $m^3$ over an area of some 2 ha (65 acre-feet), was constructed at Venissieux, France, in 1975 (Chocat et al., 1999). Inflow to the basin first passed through a sand trap before entering the basin, in which the base was covered with a geotextile and 400 mm (1.3 ft) sand. In 1988 the basin was divided into two sections and in 1990 the geotextile and sand layer were renewed. The basin now comprises sand traps, a settling basin of 1.1 ha (2.7 acres), from which flow passes through a flow regulator and hydrocarbon separator before entering an infiltration basin of 0.9 ha (2.2 acres). The Venissieux catchment is 380 ha (940 acres) of mixed industrial, agricultural and residential land use. Groundwater is 3 to 5 m (9.8 to 16 ft) below the base of the basin. Water samples have been collected at the inlets to the basin; at the outlet from the settling basin and in the infiltration basin, before and after the hydrocarbon separator.

In addition, soil samples were collected and cores obtained through the base of the infiltration basin to a depth of 1.5 m (4.9 ft). The settling basin has achieved decreases in suspended solids of between 20 and 70 percent. Examination of the soil cores showed that solids less than 2 mm (0.079 in.) particle size were seriously contaminated by heavy metals and mineral oils, although this decreased with depth until between the base of the geotextile and 0.5 m (1.6 ft) into the subgrade the pollution was slight, despite the 20 years of infiltration. No data was reported on the possible impact on groundwater quality.

### E1-2.6.7 Maintenance

Table E1-23 outlines the main maintenance requirements of an infiltration basin.

**Table E1-23 Typical Maintenance Activities For Infiltration Basins (after WMI, 1997)**

| Activity | Schedule |
|---|---|
| Inspect facility for signs of wetness or damage to structures<br>Note eroded areas.<br>If dead or dying grass on the bottom is observed, check to ensure that water percolates 2–3 days following storms.<br>Note signs of petroleum hydrocarbon contamination and handle properly. | Semi-annual inspection |
| Mow and remove litter and debris.<br>Stabilize eroded banks. | Standard maintenance (as needed) |

| Activity | Schedule |
|---|---|
| Repair undercut and eroded areas at inflow and outflow structures. | |
| Disc or otherwise aerate bottom.<br>De-thatch basin bottom. | Annual maintenance |
| Scrape bottom and remove sediment. Restore original cross-section and infiltration rate.<br>Seed or sod to restore ground cover. | 5-year maintenance |
| Remove sediment from the forebay. | When the accumulated sediment volume exceeds 50% of forebay capacity. |

E1-2.6.8 Cost

Infiltration basins are relatively cost-effective practices because little infrastructure is needed when constructing them. One study estimated the total construction cost at about $2 / $ft^3$ ($71 / $m^3$) (adjusted for inflation) of storage for a 0.25-acre basin (0.01 ha) (SWRPC, 1991). As with other BMPs, these published cost estimates may deviate greatly from what might be incurred at a specific site. For instance, Caltrans spent about $18 / $ft^3$ (640 / $m^3$) for the two infiltration basins constructed in southern California, each of which had a water quality volume of about 0.34 acre-feet (420 $m^3$).

Infiltration basins typically consume about 2 to 3 percent of the site draining to them, which is relatively small. Maintenance costs are estimated at 5 to 10 percent of construction costs.

One cost concern associated with infiltration practices is the maintenance burden and longevity. If improperly maintained, infiltration basins have a high failure rate. Thus, it may be necessary to replace the basin with a different technology after a relatively short period of time.

Maintenance costs of infiltration basins consist almost exclusively of vegetation management, unless clogging occurs.

E1-2.6.9 Research Needs

Significant uncertainty exists as to the fundamental feasibility of the infiltration approach. The basic issue is whether the temporary but deep ponding in basins makes clogging inevitable (Schueler, 1990). Other uncertainties include:

- Pretreatment system that can assure basin longevity

- The accuracy of current field methods for estimating long-term local soil infiltration rates

- The actual monitored pollutant removal performance of a fully operational basin, particularly with respect to soluble nutrients

E1-2.6.10 References

Caltrans, 2002. *BMP Retrofit Pilot Program Proposed Final Report,* CTSW-RT-01 -050, California Department of Transportation, Sacramento, CA (in press).

Construction Industry Research and Information Association (CIRIA), 2000. *C522 Sustainable Urban Drainage Systems - Design Manual for England and Wales,* London, SW1P 3AU.

Construction Industry Research and Information Association (CIRIA), 1999. *Control of Pollution from Highway Drainage Discharge. Report 142,* London, SW1P 3AU.

Galli, J. ,1992. *Analysis of Urban BMP Performance and Longevity in Prince George's County, Maryland.* Metropolitan Washington Council of Governments, Washington, DC.

Hilding, K., 1996. Longevity of Infiltration Basins Assessed in Puget Sound. *Watershed Protection Techniques* 1(3):124-125.

Mays, L.W., 2001 *Stormwater Collection Systems Design Handbook.* McGraw-Hill, Two Penn Plaza, New York.

Maryland Department of the Environment (MDE). 2000. *Maryland Stormwater Design Manual.* [http://www.mde.state.md.us/environment/wma/stormwatermanuall. Accessed May 22, 2001.

Nightingale, H.I., 1975. "Lead, Zinc, and Copper in Soils of Urban Storm-Runoff Retention Basins," *American Water Works Assoc. Journal.* vol. 67, pp 443-446.

Nightingale, H.I., 1987a. "Water Quality beneath Urban Runoff Water Management Basins," *Water Resources Bulletin*, vol. 23, pp 197-205.

Nightingale, H.I., 1987b. "Accumulation of As, Ni, Cu, and Pb in Retention and Recharge Basin Soils from Urban Runoff," *Water Resources Bulletin*, vol. 23, pp 663-672.

Nightingale, H.I., 1987c. "Organic Pollutants in Soils of Retention/Recharge Basins Receiving Urban Runoff Water," *Soil Science*, vol. 148, pp 39-45.

Nightingale, H.I., Harrison, D., and Salo, J.E., 1985. "An Evaluation Technique for Ground-water Quality Beneath Urban Runoff Retention and Percolation Basins," *Ground Water Monitoring Review*, vol. 5, No. 1, pp 43-50.

Oberts, G. ,1994. "Performance of Stormwater Ponds and Wetlands in Winter," *Watershed Protection Techniques*, 1(2): 64-68.

Pitt, R., et al. 1994, *Potential Groundwater Contamination from Intentional and Nonintentional Stormwater Infiltration*, EPA/600/R-94/05 1, Risk Reduction Engineering Laboratory, U.S. EPA, Cincinnati, OH.

Pratt C.J., 1995. *Infiltration Drainage - Case Studies of UK Practice.* Project Report 22, Construction Industry Research and Information Association, London, SWlP 3AU.

Pratt C. J., 2001. *Sustainable Urban Drainage. A Review of Published Material on the Performance of Various SUDS Devices.* Construction Industry Research and Information Association (CIRIA). www.citia.org.uk/suds/pdf/sudsreview-jan02.pd , accessed August 19, 2002.

Schueler, T. 1987. *Controlling Urban Runoff - A Practical Manual for Planning and Designing Urban BMPs.* Metropolitan Washington Council of Governments, Washington, DC.

Schueler, T.R., and J. Lugbill, 1990. *A Current Assessment of Urban Management Practices: Techniques for Reducing Non-Point Source Pollutants in the Coastal Zone.* Department of Environmental Programs, Metropolitan Washington Council of Governments, Washington, DC.

Schroeder, R.A., 1995. *Potential for Chemical Transport Beneath a Storm-Runoff Recharge (Retention) Basin for an Industrial Catchment in Fresno, CA.* USGS Water-Resource Investigations Report 93-4140.

Southeastern Wisconsin Regional Planning Commission (SWRPC), 1991. *Costs of Urban Nonpoint Source Water Pollution Control Measures.* Southeastern Wisconsin Regional Planning Commission, Waukesha, WI.

Watershed Management Institute (WMI), 1997. *Operation, Maintenance, and Management of Stormwater Management Systems.* Prepared for U.S. Environmental Protection Agency Office of Water, Washington, DC.

U.S. EPA, 1983. *Results of the Nationwide Urban Runoff Program: Volume I - Final Report,* WH-554, Water Planning Division, Washington, DC

**Information Resources**

Center for Watershed Protection (CWP), 1997. *Stormwater BMP Design Supplement for Cold Climates.* Prepared for U.S. Environmental Protection Agency Office of Wetlands, Oceans and Watersheds. Washington, DC.

The Stormwater Management Resource Centre (SMRC S): http://www.stormwatercenter.net/, accessed August 19, 2002

Ferguson, B.K., 1994. *Stormwater Infiltration.* CRC Press, Ann Arbor, MI.

USEPA, 1983. *Results of the Nationwide Urban Runoff Program: Volume 1 – Final Report,* Report WH-554, Water Planning Division, Washington, DC.

USEPA, 1993. *Guidance to Specify Management Measures for Sources of Nonpoint Pollution in Coastal Waters.* EPA-840-B-92-002. U.S. Environmental Protection Agency, Office of Water, Washington, DC.

## E1-2.7 Infiltration Trenches / Soakaways / Filter Drains

### E1-2.7.1 Description

Infiltration trenches are shallow excavations that are filled with rubble or stone to create temporary subsurface storage for infiltrations of stormwater runoff. Some manufacturers also have developed modular plastic systems which provide greater void space than rock filled trenches and reduce the amount of excavation required to store the water quality volume. Usually a small portion of the runoff, the first flush, is diverted to the infiltration trench. The runoff gradually exfiltrates through the bottom and/or sides of the trench into the subsoil and eventually to the water table. The primary pollutant removal mechanism is via filtering through the soil. Figure E1-23 provides a schematic of a typical 'end-of pipe' system. Trench designs may be modified to include vegetative cover and other features, establishing a biofiltration area.

Infiltration trenches may be designed for complete exfiltration or partial exfiltration where the first flush of the runoff volume is routed to the trench and the remainder is bypassed and conveyed to additional BMPs.

Infiltration trenches should always be constructed with pretreatment. The use of infiltration technologies should be avoided in areas with high potential pollutant loading. In groundwater drinking supply recharge areas (Zone II and Interim Wellhead Protection Areas (IWPA)) infiltration technologies may be used for uncontaminated rooftop runoff only (Stormwater Management, 1997).

In the UK, infiltration trenches are commonly known as soakaways and these can take different forms and sizes. They are built as square or circular pits, which are filled with rubble or lined with dry-joined brickwork or pre-cast perforated concrete ring units, surrounded by granular backfill. They can also take the form of trenches that follow convenient contours (BRE, 1991). Extensive use has been made of soakaways in draining small catchments in remote and rural areas.

Filtration trenches or filter trenches (Figure E1-24), which include underdrain systems in their design, combine some of the attributes of infiltration trenches and filter systems. They are used in the UK in areas of low permeability soils with the main function of purifying water prior to conveying it downstream. Filter trenches with impermeable liners are also used in areas where the risk of groundwater contamination is high.

Perforated pipes buried within the gravel of a filter trench are often used to distribute the inflow runoff along the length of the trench (Duchene, 1992). Figure E1-25 shows a typical arrangement of this application.

### E1-2.7.2 Advantages

- Provides 100 percent reduction in the load discharged to surface waters.
- An important benefit of infiltration trenches is the approximation of pre-development hydrology during which a significant portion of the average annual rainfall runoff is infiltrated and evaporated rather than flushed directly to creeks.

- If the water quality volume is adequately sized, infiltration trenches can be useful for providing control of channel forming (erosion) and high frequency (generally less than the 2-year) flood events.

- As an underground BMP, trenches are unobtrusive and have little adverse impact on site aesthetics.

**Figure E1-23 Schematic of an Infiltration Trench as 'End-Of-Pipe' System (MDE, 2000)**

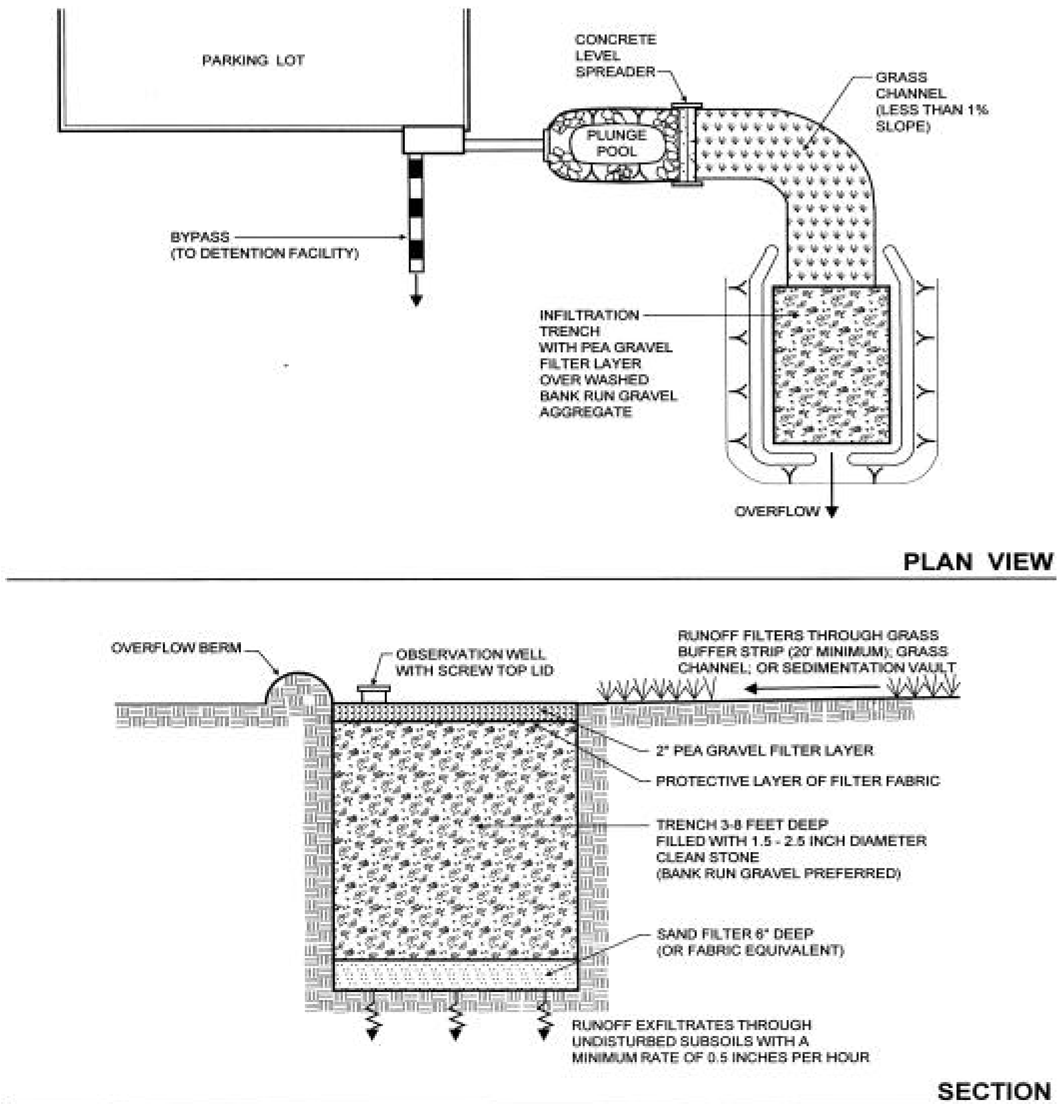


**Figure E1-24 A Filter Drain**

**Figure E1-25 Schematic of Filter Trench Using Perforated Pipes as 'End-Of Pipe' System**

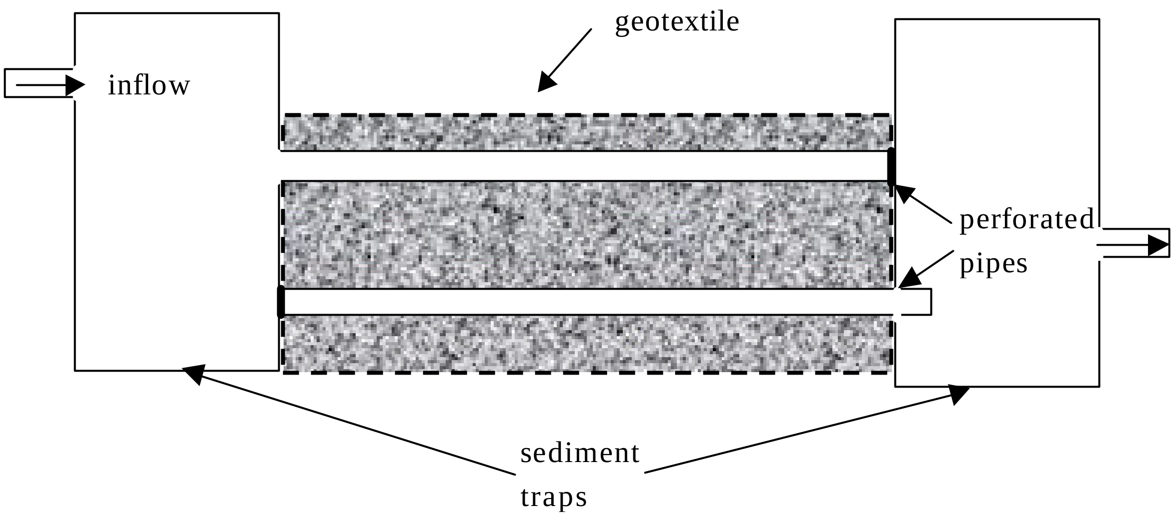
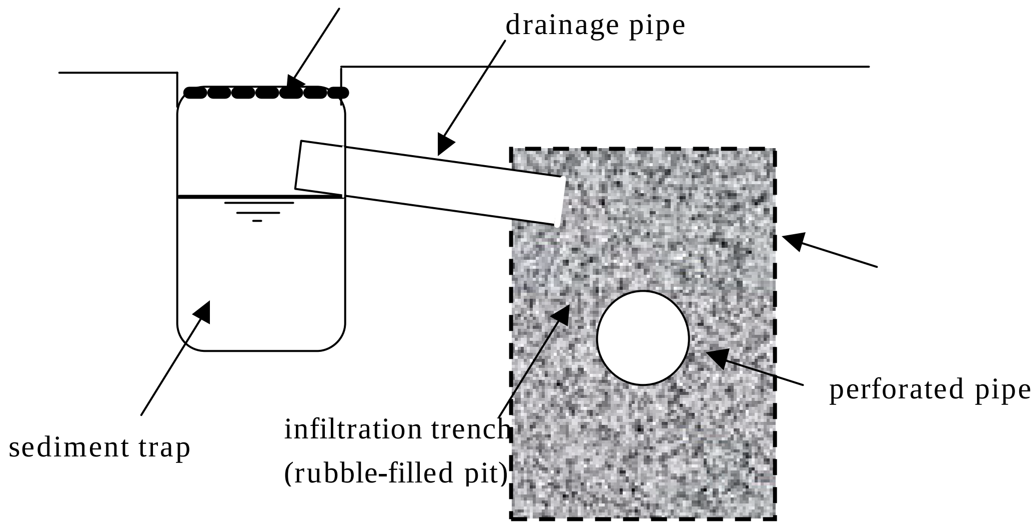


**Figure E1-26 Schematic of a Filter Drain for Road Drainage (modified from Price, 1994)**




E1-2.7.3 Limitations

Although infiltration trenches/filter drains can be a useful management practice, they have several limitations, including the following (EPA, 1997):

- High failure rate due to poor maintenance, wrong siting or high debris input.
- May not be appropriate for industrial sites or locations where spills may occur.
- Infiltration trenches are limited to relatively small catchments.
- Infiltration trenches and soakaways require a minimum soil infiltration rate or maximum emptying time depending on the location. This is not the case for filter drains and filter trenches.
- Cannot be used in area of high groundwater level or close to building foundations.
- Not suitable on fill sites or steep slopes.
- Risk of groundwater contamination.
- Upstream drainage area must be completely stabilized before construction.
- Cost of replacing filter material once blocked.
- No visual enhancement.

### E1-2.7.4 Siting Criteria

The use of infiltration trenches may be limited by a number of factors including type of native soils, climate, and high groundwater tables. Site characteristics, such as excessive slope of the drainage area, fine-grained soil types, and near surface location of the water table and bedrock, may restrict the use of infiltration trenches. Generally, infiltration trenches are not suitable for areas with relatively impermeable soils containing clay and silt or in areas with fill.

However, infiltration trenches were successfully implemented in Sweden in soil of a boulder clay of impermeable nature. The benefit from the use of the devices was in maintaining an adequate level of soil moisture, thus preventing consolidation of the soil that could result in unacceptable settlement of buildings. Holmstrand (1984) reported that up to two-thirds of the water discharged to the trenches was not passed to the outfall and that losses were due to evaporation from the soil and plant transpiration.

As stated earlier, the potential for groundwater contamination must be carefully considered, especially if the groundwater is used for human consumption or agricultural purposes. Serious concern has risen with regard to groundwater contamination from highway runoff draining into filter trenches (Price, 1994). In Germany highway runoff is generally not permitted to be directly collected in infiltration trenches due to the risk of groundwater contamination (ATV 138, 2001).

The infiltration trench is not suitable for sites where hazardous spills are likely to occur, such as industrial sites. In these areas, other BMPs that do not allow interaction with the groundwater should be considered.

Soil infiltration rate should be at least 13 mm/hr (0.5 in/hr)and the water table depth should be at least 3 m (9.8 ft) below the trench invert for complete infiltration of the stormwater volume (Caltrans, 2002). Infiltration trenches require about 0.5 to 2 percent of the catchment area for implementation.

### E1-2.7.5 Design Guidelines

Observations from Tokyo, Japan (Haneda et al., 1996) suggest that such infiltration trench / filter drain systems may operate satisfactorily with significant benefits in the reduction of direct discharges to watercourses. A key factor in continued satisfactory operation is the design, operation, and maintenance of inlet structures to the underground devices. Where roof waters discharge into sediment/debris traps, the underground, stone-filled trenches retained their infiltration capacity after 11 years of service. However, where the waters entered the system from paved surfaces, carrying considerably more silt and debris, the infiltration rates fell rapidly due to blockage (Pratt 2001).

This confirmed the previous findings by Minagawa (1990). Excavations of infiltration trenches revealed less silt within the stone fill and on the base of the trenches than expected. Extensive use was made onsite of U-shaped collecting channels and of silt traps: these played an important role in the pretreatment of materials before entry to the infiltration trenches.

UK design guidelines are outlined in the BRE 365 (1991) and the CIRIA reports C521 and C522 (2000). German guidelines are outlined in the ATV 138 (2001) and various guidelines have been published in the United States: Schueler (1987), *Stormwater Management Volume Two* (1997), EPA (1999), MDE (2000). Specific designs may vary considerably, depending on site constraints or preferences of the designer or community. There are some features, however, that should be incorporated into most infiltration trench designs and these are discussed in Table E1-24.

**Table E1-24 Design Guidelines for Infiltration Trenches**

| Design Criteria | Current Guidance / Recommendations |
|---|---|
| Hydraulic Residence Time | • Typically, filter drains are designed to store 10 mm (0.03 ft) of rainfall from the contributing area. Storage is provided below the high level outlet pipe. The low-level (under-) drain is sized on the basis that it will discharge the treatment volume in 24 hours. If the drain is near a road and the sub-grade is susceptible to waterlogging, the drainage time should be reduced. |
| Granular fill | • Specify locally available granular fill material, e.g., trench rock that is 40 to 60 mm (1.5 to 2.5) inches in diameter.<br>• A perforated or porous distributor pipe can be included in the design.<br>• For filter drains, this pipe should only be provided over the last few meters before the outlet, or adjacent to a manhole. The absence of a continuous drain for flows up to the design volume increases the attenuation and the potential biological treatment (CIRIA, 2001).<br>• With long infiltration trenches it is advisable to provide inspection tubes at regular intervals along the trench. The extreme ends of the trench should be identified by inspection tube covers or other access covers. These provide confirmation of the line and may be convenient as points of inflow for intermediate connections. (CIRIA, 2001).<br>• A high level overflow pipe should be provided for filter drains and considered for infiltration trenches. An alternative method might be to allow the drain to back up and overflow, but the route of the floodwater should be carefully designed to avoid adverse impacts, especially frequent water logging of the road sub-grade. |

| Design Criteria | Current Guidance / Recommendations |
|---|---|
| Inspection Tubes | • An inspection tube [(e.g., 225 mm (0.74 ft) perforated pipe] should be directly connected to any incoming drains to allow the free overflow of water and prevent any build-up of debris at the end of the pipe. Debris may be removed periodically from the base of the inspection tube. The tube should be slotted or perforated to minimize the build-up of hydraulic head (CIRIA, 2001). |
| Pre-treatment infrastructure | • Pretreatment structures, such as a vegetated buffer strip or water quality inlet, can increase longevity by removing sediments, hydrocarbons, and other materials that may clog the trench. In the UK, typically trapped gully pots are used in the application of road drainage and extensive use has been made of catch basins.<br>• Infiltration trenches should always be constructed with pretreatment to reduce the sediment load. BRE 365 (1991) suggests the use of wet sumps and T-piece inlets to distributor pipes in trench systems. |
| Subsoil | • The soil permeability is often difficult to assess. Some countries use data tables for each soil type, other countries require onsite investigations and the Dutch procedure allows both.<br>• A hydrogeological and a geotechnical evaluation should be undertaken to determine the suitability of the site for infiltration drainage. This is particularly important on sites where there is filled ground, as the frequent discharge of additional waters might change the soil characteristics, either chemically or structurally (CIRIA, 2001).<br>• The soil infiltration rate should be determined at the location and depth of the proposed device (CIRIA, 2001). |
| Water Table | • The site should be checked with respect to groundwater contamination vulnerability, to ensure that infiltration is an acceptable method of disposal.<br>• Infiltration systems must not adversely affect groundwater resources.<br>• Infiltration devices should not be constructed in ground where the water table reaches close to the bottom of the device at any time of the year. |
| Observation well | • Provide observation well to allow observation of drain time. |
| Trench lining | • The sides and the bottom of the trench should be lined with permeable, geotextile fabric. A layer of fine sand (clean, fine aggregate) may be substituted or used in addition on the bottom.<br>• Geotextiles should be selected to suit the surrounding soil particle size and permeability (CIRIA, 2001). |

| Design Criteria | Current Guidance / Recommendations |
|---|---|
| Infiltration Area | • Determine the effective area. In the US this is defined as the bottom surface area needed to drain the trench within 72 hr by dividing the treatment volume by the infiltration rate. In the UK this is calculated using 50% of the total depth, excluding the bottom area. The trench should be half-drained within 24 hr.<br>• Another parameter of great diversity is the effective percolation area. In many countries the bottom of the facility is assumed to be clogged after some time and is therefore not taken into consideration. Most countries use 50% of the side walls. Although this a time dependent variable it is used as a constant for the design approach. None of the procedures acknowledge the change of head (Monster, 1996). |
| Trench volume | • Determine the trench volume. This depends on whether the trench is designed to treat the first flush or the total runoff volume. The defined volume is assumed to fill the void space based on the computed porosity of the rock or other matrix. |

### E1-2.7.6 Construction Guidelines

- Soakaways and filter trenches should not be used for untreated drainage of construction sites, where runoff is likely to contain large amounts of silt, debris and other pollutants.
- Inspection tubes and/or covers should be convenient for access and the provision of an easement should be considered where multiple properties discharge to a single soakaway.
- Depending on soil type and site location, the trench may be lined with a geotextile. This should wrap over the top of the infill material to prevent ingress of soil or other surfacing material. Wherever the trench extends to the ground surface, the geotextile should be protected by 150 mm of granular or filler material.

### E1-2.7.7 Performance

Despite the extensive use that has been made of infiltration pits and trenches or soakaways there has been only limited examination of their performance (Warnaars, 1999). There has been widespread concern about the hydraulic performance of these devices, with general expectation that failure through blockage by silt and debris, would necessitate reconstruction within a limited time period (Pratt, 2001).

Infiltration trenches function similarly to rapid infiltration systems that are used in wastewater treatment. Estimated pollutant removal efficiencies from wastewater treatment performance and modeling studies shown in Table E1-25 can be used to estimate the removal of pollutants entering the subsurface. Based on these data, infiltration trenches can be expected to remove up to 90 percent of sediment, metals, coliform bacteria and organic matter, and up to 60 percent of phosphorus and nitrogen in the runoff (Schueler, 1992). Biochemical oxygen demand (BOD) removal is estimated to be between 70 to 80 percent. Lower removal rates for nitrate, chlorides and soluble metals should be expected.

**Table E1-25 Typical Removal Rates (Schueler, 1992)**

| Typical Percent Removal Rates | |
|---|---|
| Sediment | 90 |
| Total Phosphorus | 60 |
| Total Nitrogen | 60 |
| Metals | 90 |
| Bacteria | 90 |
| Organics | 90 |
| Biochemical Oxygen Demand | 75 |

There is considerable variability in the effectiveness of infiltration trenches, and proper siting, design and maintenance improves their performance.

In the USA in the late 1990s, research was undertaken on the water quality improvement capability of a partial infiltration trench at a site on the Millcreek Expressway in Cincinnati, Ohio (Sansalone, 1999). Road runoff was intercepted by a small section of porous pavement and filtered through the granular backfilled trench. Runoff could infiltrate the adjacent soil or be discharged from the filter media via a pipe, the flow from which was monitored and sampled. The heavy metal and total suspended solids removal efficiency results showed that the system was an effective trap, as shown in Table E1-26. Concern was expressed about the potential for clogging of the system and the effect this would have on the design life of 10 years.

Minagawa (1990) reporting field observations of the silting of sedimentation / infiltration chambers and of porous pipes within interconnecting infiltration trenches over 5 years at 3 residential areas in Tokyo. The infiltration capacity of the chambers was effectively reinstated after removal of 150 mm (0.5 ft) of the crushed rock and deposits at the bottom of the chambers (Pratt, 2002).

**Table E1-26 Typical removal rates (Sansalone, 1999)**

| Typical Percent Removal Rates | |
|---|---|
| Dissolved Metals: | |
| - Zinc | >95 |
| - Copper | >85 |
| - Cadmium | >80 |
| - Lead | 70-95 |
| Particulate-bound metals: | |
| - Zinc | 75-95 |
| - Copper | 85-95 |
| - Cadmium | 79-90 |
| - Lead | 85-95 |

A study of the impact of highway runoff on soil and groundwater conducted in Switzerland provides information on likely infiltration water quality below infiltration surfaces (Mikkelsen et al., 1997). Two sites were studied. At the first, highway runoff was discharged via a pipe from a curb inlet gully into a depression on the grass-covered verge (shoulder). The road carried an estimated 37,000 vehicles a day in 1993, and the drainage had been in service since 1959. High concentrations of heavy metals, a number of polynuclear aromatic hydrocarbons (PAHs) and adsorbed organically bound halogens were found in the upper 500 mm (1.6 ft) of runoff sludge and soil, but the concentrations decreased rapidly to background levels.

At the second site, highway runoff was discharged into 3 m deep soakaways at a location where traffic density was estimated at 2,300 vehicles per day. Here, the runoff sludge was some 200 mm (0.66 ft) deep and traces of contamination were not found further than another 200 mm (0.66 ft) below that level. Similar findings were reported for two soakaways investigated at Brandon, Suffolk (Pratt, 1995, 1996). High concentrations of total organic carbon and heavy metals were associated with fine, organic material accumulation in the first 400 mm (1.3 ft) of sediment in the base of the soakaways. Below 400 mm the pollutant levels appeared to approach those of background levels.

It is therefore proposed that the formation and continued presence of a layer of sludge at the base of the soakaway is important in retaining pollutants through filtration and adsorption, resulting in significant build-up of copper, zone, cadmium, PAHs and halogens in that layer (Pratt, 2002). It was concluded at the Swiss sites that 'leaching of heavy metals is limited and that contamination of potable groundwater with metals is of little practical concern within a reasonable time frame'. This conclusion holds for the PAHs, which are known to adsorb well in soil systems, and for the type of pollutants included in the organically bound halogen analysis

(Pratt, 2002). However, it is stressed by Mikkelsen that soluble components such as pesticides and de-icing salts may pass directly through infiltration systems.

A recent study at HR Wallingford looked at the hydraulic performance of a perforated concrete ring soakaway (Abbott, 2001). Observations of actual infiltration from the soakaway, occurring within 24-hour periods during different storm events, showed values of 6.4 – 8.9 percent of fill volume instead of the 50 percent recommended by BRE and CIRIA.

UK research on a typical roadside filter drain was conducted in 1999 (Schlüter, 2002). Preliminary results after 2 years of operation showed peak TSS removal rates of 75 percent but conductivity increasing by 73 percent. The conductivity rise was thought to be due to the use of roadside salts. Percentage outflow rates were found to be in the order of 40 percent and lag times varied from zero to 11.5 hours. Analysis of the most recent datasets suggests sediment break-through at the outflow of the system. Monitoring is ongoing. Water quality performance measurements are summarized in Table E1-27.

**Table E1-27 Trench Pollutant Removal Efficiency Data**

| Removal Efficiencies, % | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Study** | **TSS** | **o-phos** | **TN** | **$NO_3$** | **Metals** | **HCs** | **Type** |
| Folkes Skinner, 2002 (Hopwood Services) | 85 | | | -86 | Cu: 72 Pb: 85<br>Zn: 83 Cd: 38 | | Filter drain |
| Perry & McIntyre, 1986 | 85 | | | | Pb: 83 Zn: 81 | 70 | Filter drain |
| Winer, 2000 | | 100 | | 42 | | | Infiltration trench |
| Schueler, 1992 | 90 | 60 | 60 | | 90 | | Infiltration trench |
| Sansalone, 1999 | | | | | Cu: 85 Pb: 85<br>Zn: 85 Cd: 80 | | Infiltration trench |
| Schluter, 2002 | 75 | | | | | | Filter drain |

### E1-2.7.8 Maintenance

Longevity can be increased by careful geotechnical evaluation prior to construction and by designing and implementing an inspection and maintenance plan.

Price (1994) points out that trapping catch basins discharging to filter drains were formerly emptied every 2 months. Since the reorganization of the UK local government in 1974, the interval between gully emptying has increased to a year or more. Data published in the CIRIA

Report 134 (1995) shows gully pot filling times of approximately 4 to 6 months. Clearly at these locations an annual emptying interval would not be sufficient. There is concern that water held in traps for a long time may provide a breeding ground for bacteria and concentrate pollutants which are then carried out as a concentrated 'first flush' when it rains (CIRIA 1999, Price 1994, Butler, 1999).

A problem frequently encountered with drains and sewers is the ingress of tree roots through poor cracks in the pipes. Roots will be drawn to the location of a soakaway. It is unclear whether this is a benefit or a problem as the roots may provide additional pathways for water to enter the soil, and the tree will take up water itself.

The following points should be considered (EPA, 1999):

- to confirm that the desired drain time has been obtained (BRE 356, 1991).
- Inspections and maintenance for standing water, trash and debris, and sediment accumulation.
- Observe drain time for the design storm after completion or modification of the facility An aggressive maintenance and inspections schedule is advisable to ensure longevity of the filtration /infiltration device. The schedule should vary with location and system used.
- Remove accumulated trash and debris in the trench as often as practicable.
- If sediment is visible on top of the trench, remove top layer of trench, silt, filter fabric and stone; wash stone and reinstall fabric and stone into trench.
- Inspect for standing water after main rainfall events.

Typical maintenance activities are summarized in Table E1-28.

**Table E1-28 Typical Maintenance Activities for Infiltration Trenches/Soakaways**

| Activity | Schedule |
|---|---|
| • Removal of debris from the floor of the inspection tube or chamber<br>• Cleaning of any filters on downpipes<br>• Emptying of debris from any vortex filter trap or catchpit.<br>• Removal and washing of exposed stones on the trench surface.<br>• Trimming of any roots that may be causing blockages. | Annually<br>(semi-annually the first year) |
| • Replacement of the geotextile and rock fill will be necessary if the device becomes blocked with silt (> 5 years). | As needed (infrequent) |

## E1-2.7.9 Cost

### *E1-2.7.9.1 Construction Costs*

Construction costs include clearing, excavation, placement of the filter fabric and stone, installation of the monitoring well, and establishment of a vegetated buffer strip. Additional costs include planning, geotechnical evaluation, engineering and permitting. Typical construction costs, including contingency and design costs, are about \$5 per $ft^3$ (\$180/$m^3$) of stormwater treated (SWRPC, 1991; Brown and Schueler, 1997).

The Southeastern Wisconsin Regional Planning Commission (SWRPC, 1991) has developed cost curves and tables for infiltration trenches based on 1989 dollars. The 1993 construction cost for a relatively large infiltration trench (i.e., 1.8 meters (6 feet) deep and 1.2 meters (4 feet) wide with a 68 cubic meter (2,400 cubic feet) volume) ranged from \$8,000 to \$19,000. A smaller infiltration trench [i.e., 0.9 meters (3 feet) deep and 1.2 meters (4 feet) wide with a 34 cubic meter (1,200 cubic feet) volume] was estimated to cost from \$3,000 to \$8,500.

The USEPA, 1999 provides the following table:

| SUD | Construction Cost Equation | Construction Costs | Typical design contingency and other capital costs (30% of construction costs) | Annual maintenance costs (12 % of construction costs) |
|---|---|---|---|---|
| Infiltration Trench | 3.9* water Quality Volume + 2900 | \$47, 100 | \$14, 100 | \$5, 600 |

SEWRP 1991, Schuler 1987, USEPA 1993

Trench-type soakaways are less expensive than the alternative forms of ring unit soakaway, since the volumes of excavation and granular material backfill alone are similar for both types without even taking into consideration the cost of ring units. (Pratt C.J., 1996)

### *E1-2.7.9.2 Maintenance Costs*

Maintenance costs include buffer strip maintenance and trench inspection and rehabilitation. SEWRPC (1991) has also developed maintenance costs for infiltration trenches. Based on the above examples, annual operation and maintenance costs would average \$700 for the large trench and \$325 for the small trench. Typically, annual maintenance costs are approximately 5 to 10 percent of the capital cost (Schueler, 1987). Trench rehabilitation, may be required every 5 to 15 years. Cost for rehabilitation will vary depending on site conditions and the degree of clogging. Trench rehabilitation is estimated to cost between 15 and 20 percent of the original capital costs. (SWRPC, 1991).

### E1-2.7.10 Research Needs

#### *E1-2.7.10.1 Siting Criteria*

There are substantial differences in siting criteria applied in Germany, UK, and US regarding the potential for groundwater contamination from certain land uses such as highways and industrial sites despite research that indicates little transport of pollutants in the subsurface. Some additional research may be warranted to determine if there is a substantial threat from spills or runoff with relatively high pollutant concentrations.

#### *E1-2.7.10.2 Design Guidelines*

Design methods vary significantly in how the required size of the trench is calculated based on permeability and surface available for infiltrations. Confirmation of modeling results with respect to effective drainage area (bottom and sides of trench) and effect of blockage of the bottom of the trench would be useful.

#### *E1-2.7.10.3 Performance*

The potential impact on groundwater contamination remains a concern despite evidence that indicates rapid attenuation of pollutant concentrations in the subsurface. Clearly the amount of attenuation is strongly dependent on the characteristics of the soil, so additional evaluation of the performance in sandy soils near the water table may be warranted.

Little of the available performance data is from cold weather climates, so there is a large uncertainty in their performance in subfreezing weather and during snowmelt runoff conditions. Additionally, an investigation of possible groundwater contamination by soluble substances de-icing salts is required.

#### *E1-2.7.10.4 Construction Guidelines and Cost*

The wide range of cost data indicates that much more information is needed in this area. Do the differences in cost reflect different regulatory requirements, differences in construction techniques, or perhaps site suitability? Identifying the primary elements that drive construction costs upward is a high priority and could help reduce the initial cost associated with this BMP / SUDS.

*E1-2.7.10.5 Maintenance Guidelines and Cost*

Maintenance requirements for the trench itself are generally minimal, with the most significant maintenance needed for the pretreatment systems (buffer strip, etc.). A major question is whether the cost of reconstruction of the trench every 10 years will be substantially the same as the original construction cost.

### E1-2.7.11 References

Abbott C.L. and Comino-Mateos L., 2001. “”In Situ Performance Monitoring of an Infiltration Drainage System and Field Testing of Current Design Procedures,” *Journal CIWEM*, 15(3), pp. 198-202.

Alleghenies Technische Vorschriften (ATV), 2001. *Report 138 Planung, Bau und Betrieb von Anlagen zur Versickerung von Niederschlagswasser*, Gesellschaft zur Foerderung der Abwassertechnik e.V., Hennef, Germany.

Brown, W., and T. Schueler, 1997. *The Economics of Stormwater BMPs in the Mid-Atlantic Region.* Prepared for the Chesapeake Research Consortium, Edgewater, MD, by the Center for Watershed Protection, Ellicott City, MD.

Building Research Establishment (BRE), 1991. *Digest 365 Soakaway Design.*

Butler, D. and Memon, F., 1999. Dynamic Modeling Of Roadside Gully Pots During Wet Weather, *Water Research* vol. 33, no. 15, pp 3364-3372, Elsevier Science Ltd.

Caltrans, 2002. *BMP Retrofit Pilot Program Proposed Final Report*, CTSW-RT-01-050, California Department of Transportation, Sacramento, CA (in press)

Construction Industry Research and Information Association (CIRIA). 2000. *Sustainable Urban Drainage Systems - Design Manual for Scotland and Northern Ireland Report C521*, London, SW1P 3AU.

Construction Industry Research and Information Association (CIRIA), 2000. *C522 Sustainable Urban Drainage Systems - Design Manual for England and Wales*, London, SW1P 3AU.

Construction Industry Research and Information Association (CIRIA), 1995. *Sediment Management In Urban Drainage Catchments. Report 134*, London, SW1P 3AU.

Construction Industry Research and Information Association (CIRIA), 1999**.** *Control of Pollution From Highway Drainage Discharge. Report 142,* London, SW1P 3AU.

Duchene, M. McBean, E. , 1992. Discharge characteristics of perforated pipe for use in infiltration trenches. *Water Resource Bulletin*. American Water Resource Association, vol. 28, no. 3, pp 517-524.

Duchene, M. McBean, E. Thomson, N.R., 1994. Modeling of Infiltration from Infiltration Trenches for Stormwater Control. *Journal of Water Resources Planning and Management*, *v*ol. 120, no. 3, pp 276-293. ISBN / ISSN: 0733-9496

Galli, J., 1992. *Analysis of Urban BMP Performance and Longevity in Prince George's County, Maryland*. Metropolitan Washington Council of Governments, Washington, DC.

Haneda M., Kagawa A. and Maruyama K., 1996**. "**Study on Durability of Underground Infiltration Facilities Through Long-Term Survey," Proceedings, 7th International Conference on Urban Storm Drainage, Hannover, Germany, pp 599-604.

Maryland Department of the Environment (MDE), 2000. *Maryland Stormwater Design Manual* [http://www.mde.state.md.us/environment/wma/stormwater-manual]. Accessed August 19, 2002.

Minagawa, K., 1990**. "**The Stormwater Infiltration System in Housing Complexes and the Follow-Up Survey." *In: Drainage Systems and Runoff Reduction*, Eds. Y. Iwasa and T. Sueishi, (Proceedings, 5th International Conference on Urban Storm Drainage, vol. 2), Osaka, Japan, pp 771-776.

Monster, N.J., Leeflang M.J., 1996. *On the Design of Percolation Facilities*. Communication of the Department of Sanitary Engineering and Water Management. Delft University of Technology, Netherlands ISSN 0169-6246.

Pratt, C. J., 2001. *Sustainable Urban Drainage. A Review of Published Material on the Performance of Various SUDS Devices.* Construction Industry Research and Information Association (CIRIA). www.ciria.org.uk/suds/pdf/suds_review-jan02.pdf, accessed August 19, 2002.

Price, M., 1994. Drainage from Roads and airfields to soakaways: groundwater pollutant or valuable recharge? *Journal of the Institution of Water and Environmental Management*, Vol. 8, pp 468-479.

Sansalone, J.J., Smithson, J.A., and Koran, J.M., 1998. "Development and Testing of a Partial Exfiltration Trench for *In-Situ* Treatment of Highway Drainage," *Transportation Research Record*, vol. 1647, pp 34-42.

Schlüter, W. Spitzer A, Jefferies C. 2002. Performance of Three Sustainable Urban Drainage Systems in East Scotland. Paper for 9th International Conference on Urban Drainage, Portland OR.

Schueler, T.R. 1987. *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*. Metropolitan Washington Council of Governments, Washington, DC.

Schueler, T.R., 1992. *A Current Assessment of Urban Best Management Practice*s. Metropolitan Washington Council of Governments. Southeastern Wisconsin Regional Planning Commission.

Schueler, T.R., and J. Lugbill, 1990. *A Current Assessment of Urban Management Practices: Techniques for Reducing Non-Point Source Pollutants in the Coastal Zone*. Department of Environmental Programs, Metropolitan Washington Council of Governments, Washington, DC.

Southeastern Wisconsin Regional Planning Commission (SWRPC), 1991. *Costs of Urban Nonpoint Source Water Pollution Control Measures*. Southeastern Wisconsin Regional Planning Commission, Waukesha, WI.

Stormwater Management. Massachusetts Department of Environmental Protection, 1997. *Stormwater Management Volume Two: Stormwater Technical Handbook*, Boston, MA.

Todorovic Z., Butler D., Maksimovic C., Prokrajac D., Deletic A., 2002. "Simulation of the Long-Term Hydraulic Behaviour of Stormwater Infiltration Trenches." Paper to be published in *Urban Water Journal*, Elsevier Science.

United States Environmental Protection Agency (EPA), 1999. *National Menu of Best Management Practices for Stormwater Phase* [http://cfpub.epa.gov/npdes/storm-water/menuofbmps-/post_14.cfm] and [http://www.epa.gov/owm/mtb/infltrenc-.pdf] accessed August 22, 2002.

Warnaars, E., Larsen, A.V., Jacobsen, P. and Mikkelsen, P.S., 1999. "Hydrologic Behaviour Of Stormwater Infiltration Trenches In A Central Urban Area During 23/4 Years Of Operation," *Wat. Sci. Tech*., 39(2), pp 217-224.

## E1-2.8 Bioretention

### E1-2.8.1 Description

Bioretention functions as a soil and plant-based filtration device that removes pollutants through a variety of physical, biological, and chemical treatment processes. As shown in the Stormwater Technology Fact Sheet: Bioretention (EPA, 1999), Figure E1-27, displays runoff conveyed as sheet flow to the treatment area. This bioretention area consists of a grass buffer strip, sand bed, ponding area, organic layer or mulch layer, planting soil, and plants. The runoff's velocity is reduced by passing over or through a sand bed and subsequently distributed evenly along a ponding area. The ponding area, a surface organic layer and/or ground cover and the underlying planting soil, is graded and its center depressed. As the water gradually infiltrates the bioretention area or is evapotranspired, it is ponded to a depth of 6 in (15 cm). Note that the bioretention area is graded to divert runoff in excess of the design storm volume. Exfiltration of the stored water in the bioretention area planting soil into the underlying soils occurs over a period of days.

Each component of the bioretention area is designed to perform a specific function as indicated in Table E1-29.

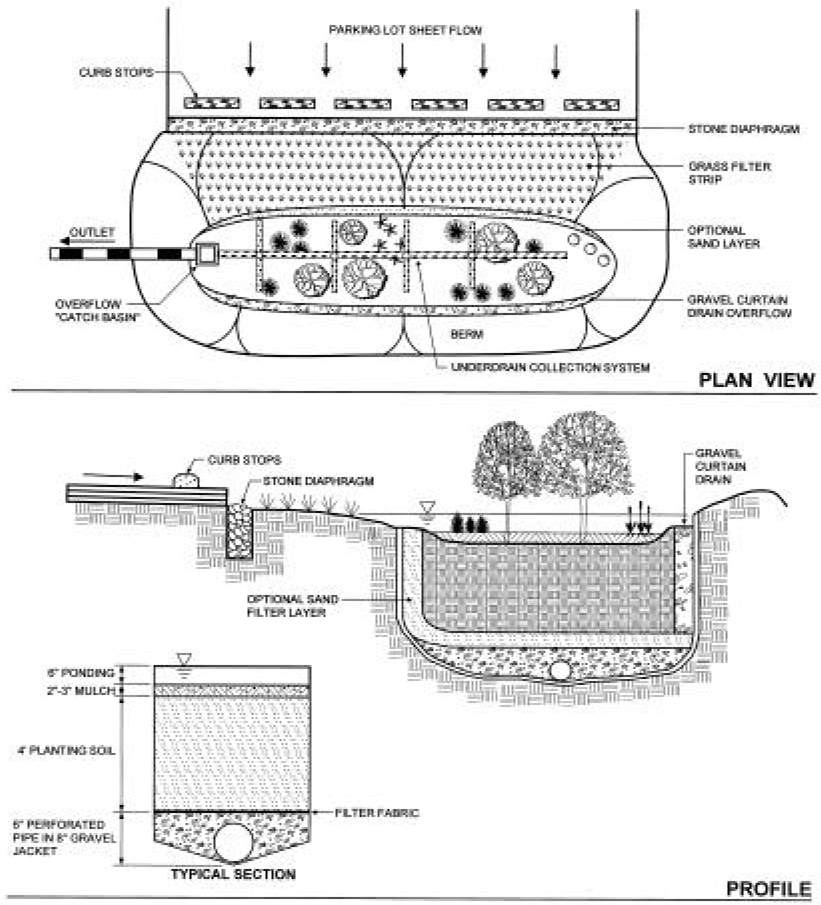
The basic bioretention area can be customized to accommodate specific needs. For example, the City of Alexandria, VA modified the bioretention BMP because impervious subsoils and marine clays prevented complete infiltration in the soil system. The redesigned bioretention BMP, which includes an underdrain within the sand bed that collects the infiltrated water and discharges it to a downstream sewer system, acts more as a filter that discharges treated water than as an infiltration device. Design modifications that will potentially include both aerobic and anaerobic zones in the treatment area are being reviewed. The anaerobic zone will promote denitrification.

**Table E1-29 Bioretention Component Functions**

| | |
|---|---|
| Grass buffer strip | Reduces runoff velocity and filters particulates from runoff. |
| Sand bed | Reduces velocity, filters particulates, and spreads flow over the length of the bioretention area. Aeration and drainage of the planting soil are provided by the 18 inch (0.5 meter) deep sand bed. |
| Ponding area | Provides temporary storage location for runoff prior to its evaporation or infiltration. Particulates not filtered out by the grass filter strip or the sand bed settle within the ponding area. |
| Organic/Mulch layer | Filters pollutants and provides an environment conducive to the growth of microorganisms, which degrade petroleum-based products and other organic material. This layer acts in a similar way to the leaf litter in a forest and prevents the erosion and drying of underlying soils. The maximum sheet flow velocity prior to erosive conditions is 0.9 m/s (3 ft/s). |
| Planted ground cover | Reduces the potential for erosion as well, slightly more effectively than mulch. The maximum sheet flow velocity prior to erosive conditions is 0.3 m/s (1 ft/s) for planted ground cover. |
| Clay in planting soil | Provides adsorption sites for hydrocarbons, heavy metals, nutrients and other pollutants. |
| Voids in planting soil | Provides stormwater storage. The stored water and nutrients in the water and soil are then available to the plants for uptake. |

A number of laboratory and field experiments have been conducted by the University of Maryland in conjunction with Prince George's County Department of Environmental Resources and the National Science Foundation in order to quantify the effectiveness of bioretention cells in terms of pollutant removal (Davis et al., 2001). A web site dedicated to this work can be found at http://www.ence.umd.edu/~apdavis/Bioret.htm.

**Figure E1-27 Schematic of a Bioretention Area (MDE, 2000)**



### E1-2.8.2 Advantages

- Bioretention provides stormwater treatment that enhances the quality of downstream water bodies by temporarily storing runoff in the BMP and releasing it over a period of four days to the receiving water (EPA, 1999).
- The vegetation provides shade and wind breaks, absorbs noise, and improves an area's landscape.

### E1-2.8.3 Limitations

- The bioretention BMP is not recommended for areas with slopes greater than 20% or where mature tree removal would be required since clogging may result, particularly if the BMP receives runoff with high sediment loads (EPA, 1999).
- Bioretention is not a suitable BMP at locations where the water table is within 6 ft of the ground surface and where the surrounding soil stratum is unstable.
- In cold climates the soil may freeze, preventing runoff from infiltrating into the planting soil.

### E1-2.8.4 Siting Criteria

Bioretention BMPs are generally used to treat stormwater from impervious surfaces at commercial, residential, and industrial areas (EPA, 1999). Implementation of bioretention for stormwater management is ideal for median strips, parking lot islands, and swales. Moreover, the runoff in these areas can be designed to either divert directly into the bioretention area or convey into the bioretention area by a curb and gutter collection system.

The best location for bioretention areas is upland from inlets that receive sheet flow from graded areas and at areas that will be excavated (EPA, 1999). In order to maximize treatment effectiveness, the site must be graded in a way that minimizes erosive conditions as sheet flow is conveyed to the treatment area. Locations where a bioretention area can be readily incorporated into the site plan without further environmental damage are preferred. Furthermore, to effectively minimize sediment loading in the treatment area, bioretention only should be used in stabilized drainage areas.

The layout of the bioretention area is determined after site constraints such as location of utilities, underlying soils, existing vegetation, and drainage are considered (EPA, 1999). Sites with loamy sand soils are especially appropriate for bioretention because the excavated soil can be backfilled and used as the planting soil, thus eliminating the cost of importing planting soil.

The use of bioretention may not be feasible given an unstable surrounding soil stratum, soils with a clay content greater than 25 percent, a site with slopes greater than 20 percent, and/or a site with mature trees that would be removed during construction of the BMP.

Bioretention can be designed to be off-line or on-line of the existing drainage system (EPA, 1999). The drainage area for a bioretention area should be between 0.25 and 1.0 acres. Larger drainage areas may require multiple bioretention areas. Stabilized areas may erode when flow velocities are greater than 1.5 m/s (5 ft/s). In Maryland, such a flow generally occurs with a 10-year storm at 0.4 ha (1 acre) commercial or residential sites. The designer should determine the potential for erosive conditions at the site.

### E1-2.8.5 Design Guidelines

Prince George's County DER has specified bioretention design details in a document entitled *Design Manual for the Use of Bioretention in Storm Water Management* (PGDER, 1993). Design specifications were derived from research on soil adsorption capacities and rates, water balance, plant pollutant removal potential, plant adsorption capacities and rates, and maintenance requirements. A case study was performed using the specifications at three commercial sites and one residential site in Prince George's County, MD.

**Table E1-30 Bioretention Design Guidelines**

| | |
|---|---|
| Hydraulic Sizing | • The size of the bioretention area, which is a function of the drainage area and the runoff generated from the area, should be 5 to 7 percent of the drainage area multiplied by the rational method runoff coefficient, "c," determined for the site. The 5 % specification applies to a bioretention area that includes a sand bed and 7 % applies to an area without one.<br>• To provide adequate storage and prevent water from standing for excessive periods of time, the ponding depth of the bioretention area should not exceed 15 cm (6 in). Water should not be left to stand for more than 72 hours. It may be necessary to restrict the type of plants that can be used may be necessary due to the water intolerance of some plants. Further, if water is left is standing for longer than 72 hours mosquitoes and other insects may start to breed. |
| Planting Dimensions | • The *recommended* minimum dimensions of the bioretention area are 4.6m (15 ft) ) wide by 12.2 m (40 ft) long, where the minimum width allows enough space for a dense, randomly-distributed area of trees and shrubs to become established. This replicates a natural forest and creating a microclimate, thereby enabling the bioretention area to tolerate the effects of heat stress, acid rain, runoff pollutants, and insect and disease infestations which landscaped areas in urban settings typically are unable to tolerate. The *preferred* width is 7.6 m (25 ft), with a length of twice the width. Essentially, any facilities wider than 6.1 m (20 ft) should be twice as long as they are wide, which promotes the distribution of flow and decreases the chances of concentrated flow. |

| | |
|---|---|
| Soil | • Appropriate planting soil should be backfilled into excavated bioretention area. Planting soils should be sandy loam, loamy sand, or loam texture with a clay content from 10 to 25%.<br>• Planting soil should be 10 cm (4 in) deeper than the bottom of the largest root ball and 1.2 m (4 ft) altogether. This depth will provide adequate soil for the plants' root systems to become established, prevent plant damage due to severe wind, and provide adequate moisture capacity. Most sites will require excavation in order to obtain the recommended depth.<br>• Generally the soil should have infiltration rates greater than 1.2 cm (0.5 in) per hour, which is typical of sandy loams, loamy sands, or loams.<br>• The pH of the soil should range between 5.5 and 6.5, where pollutants such as organic nitrogen and phosphorus can be adsorbed by the soil and microbial activity can flourish. Additional requirements for the planting soil include a 1.5 to 3 percent organic content and a maximum 500 ppm concentration of soluble salts. Furthermore, the criteria for magnesium, phosphorus, and potassium are 39 kg/ac<br>• (35 lb/ac), 112 kg/ac (100 lb/ac), and 95 kg/ac (85 lb/ac), respectively.<br>• Soil tests should be performed for every 382 $m^3$ (500 $y^3$) of planting soil, except for pH and organic content tests, which are required only once per bioretention area (EPA, 1999). |
| Plants | • Since high canopy trees may be destroyed during maintenance, the bioretention area should be vegetated to resemble a terrestrial forest community ecosystem that is dominated by understory trees. Also, the bioretention area should have discrete soil zones as well as a mature canopy and a distinct sub-canopy of understory trees, a shrub layer, and herbaceous ground covers.<br>• Three species each of both trees and shrubs are recommended to be planted at a rate of 2500 trees and shrubs per hectare (1000 per acre). For instance, a 4.6m (15 ft) by 12.2m (40 ft) bioretention area 55.8 $m^2$ (600 $ft^2$) would require 14 trees and shrubs. The shrub-to-tree ratio should be 2:1 to 3:1. On average, the trees should be spaced 3.6 m (12 ft) apart and the shrubs should be spaced 2.4 m (8 ft) apart.<br>• Native species tolerant to pollutant loads and varying wet and dry conditions should be used in bioretention area. These species can be determined from several published sources, including Native Trees, Shrubs, and Vines for Urban and Rural America (Hightshoe, 1988).<br>• The designer should assess aesthetics, site layout, and maintenance requirements when selecting plant species. Adjacent non-native invasive species should be identified and the designer should take measures, such as providing a soil breach to eliminate the threat of these species invading the bioretention area. Regional landscaping manuals should be consulted to ensure that the planting of the bioretention area meets the landscaping requirements established by the local authorities.<br>• The designers should evaluate the best placement of vegetation within the bioretention area. Plants should be placed at irregular intervals to replicate a natural forest. Trees should be placed on the perimeter of the area to provide shade and shelter from the wind. Trees and shrubs can be sheltered from damaging flows if they are placed away from the path of the incoming runoff. In cold climates, species that are more tolerant to cold winds, such as evergreens, should be placed in windier areas of the site. |
| Final cover | • Following placement of the trees and shrubs, the ground cover and/or mulch should be established. Mulch depths should be kept below 7.6 cm (3 in) because more would interfere with the cycling of carbon dioxide and oxygen between the soil and the atmosphere. Ideally mulch should be aged for one year (minimum of at least six months) and applied uniformly over the site. |

### E1-2.8.6 Performance

Bioretention removes stormwater pollutants through physical and biological processes, including adsorption, filtration, plant uptake, microbial activity, decomposition, sedimentation and volatilization (EPA, 1999). Adsorption is the process whereby particulate pollutants attach to soil (e.g., clay) or vegetation surfaces. Adequate contact time between the surface and pollutant must be provided for in the design of the system for this removal process to occur. Thus, the infiltration rate of the soils must not exceed those specified in the design criteria or pollutant removal may decrease. Pollutants removed by adsorption include metals, phosphorus, and hydrocarbons. Filtration occurs as runoff passes through the bioretention area media, such as the sand bed, ground cover and planting soil.

Common particulates removed from stormwater include particulate organic matter, phosphorus, and suspended solids. Biological processes that occur in wetlands result in pollutant uptake by plants and microorganisms in the soil. Plant growth is sustained by the uptake of nutrients from the soils, with woody plants locking up these nutrients through the seasons. Microbial activity within the soil also contributes to the removal of nitrogen and organic matter. Nitrogen is removed by nitrifying and denitrifying bacteria, while aerobic bacteria are responsible for the decomposition of the organic matter. Microbial processes require oxygen and can result in depleted oxygen levels if the bioretention area is not adequately aerated. Sedimentation occurs in the swale or ponding area as the velocity slows and solids fall out of suspension. The removal effectiveness of bioretention has been studied during field and laboratory studies conducted by the University of Maryland (Davis et al., 1998). During these experiments, synthetic stormwater runoff was pumped through several laboratory and field bioretention areas to simulate typical storm events in Prince George's County, MD. Removal rates for heavy metals and nutrients are shown in Table E1-31.

Results for both the laboratory and field experiments were similar for each of the pollutants analyzed. Doubling or halving the influent pollutant levels had little effect on the effluent pollutants concentrations (Davis et al., 1998). The microbial activity and plant uptake occurring in the bioretention area will likely result in higher removal rates than those determined for infiltration BMPs.

**Table E1-31 Laboratory and Estimated Bioretention (Davis et al., 1998; PGDER, 1993)**

| Pollutant | Removal Rate, % |
|---|---|
| Total Phosphorus | 70-83 |
| Metals (Cu, Zn, Pb) | 93-98 |
| TKN | 68-80% |
| Total Suspended Solids | 90% |
| Organics | 90% |
| Bacteria | 90% |

E1-2.8.7 Maintenance

The primary maintenance requirement for bioretention areas is that of inspection and repair or replacement of the treatment area's components. Generally, this involves nothing more than the routine periodic maintenance that is required of any landscaped area. Native plant species should have been selected for use in the bioretention cell, reducing fertilizer, pesticide, water, and overall maintenance requirements. Bioretention system components should blend over time through plant and root growth, organic decomposition, and the development of a natural soil horizon. These biologic and physical processes over time will lengthen the facility's life span and reduce the need for extensive maintenance. Maintenance requirements are summarized in Table E1-32.

E1-2.8.8 Costs

*E1-2.8.8.1 Construction Costs*

Construction cost estimates for a bioretention area are slightly greater than those for the required landscaping for a new development (EPA, 1999). A general rule of thumb (Coffman, 1999) is that residential bioretention areas average about \$32 to \$43 per $m^2$ (\$3 to \$4 per $ft^2$), depending on soil conditions and the density and types of plants used. Commercial, industrial and institutional site costs can range between \$110 and \$430 per $m^2$ (\$10 to \$40 per $ft^2$), based on the need for control structures, curbing, storm drains and underdrains.

**Table E1-32 Typical Maintenance for Bioretention Areas**

| Activity | Schedule |
|---|---|
| • Evaluate the trees and shrubs and subsequently remove any dead or diseased vegetation (EPA, 1999). Diseased vegetation should be treated as needed using preventive and low-toxic measures to the extent possible. | Bi-annually |
| • Replace mulch when erosion is evident or when the site begins to look unattractive. Specifically, the entire area may require mulch replacement every two to three years, although spot mulching may be sufficient when there are random void areas. Mulch replacement should be done during the spring and the old mulch should be removed prior to distribution of the new mulch. | As necessary or every 2-3 years |
| • Apply an alkaline product, such as limestone, to counteract soil acidity resulting from slightly acidic precipitation and runoff. Before the limestone is applied, the soils and organic layer should be tested to determine the pH and therefore the quantity of limestone required. When levels of pollutants reach toxic levels which impair plant growth and the effectiveness of the BMP, soil replacement may be required (PGDER, 1993). | 1-2 times per year |
| • New Jersey's Department of Environmental Protection states in their bioretention systems standards that accumulated sediment and debris removal (especially at the inflow point) will normally be the primary maintenance function. Other potential tasks include replacement of dead vegetation, soil pH regulation, erosion repair at inflow points, mulch replenishment, unclogging the underdrain, and repairing overflow structures. | As needed |
| • Replace soils if cation exchange capacity is significantly reduced. | Possibly every 5-10 years. |

Bioretention areas recently constructed in Prince George's County, MD cost approximately $500 (EPA, 1999). These units are rather small (400 $ft^2$ or 37 $m^2$) and their cost relatively low. The cost estimate includes the cost of excavating 0.6 to 1 m (2 to 3 ft) and vegetating the site with 1 to 2 trees and 3 to 5 shrubs. The cost of planting soil, which is not included in the estimate, increases the overall cost for a bioretention area. Retrofitting a site typically costs more, averaging $6,500 per bioretention area. The higher costs are attributed to the demolition of existing concrete, asphalt, and existing structures and the replacement of fill material with planting soil. The costs of retrofitting a commercial site in Maryland, Kettering Development, with 15 bioretention areas were estimated at $111,600.

In any bioretention area design, the cost of plants varies substantially and can account for a significant portion of the expenditures. While these cost estimates are slightly greater than those of typical landscaping treatment (due to the increased number of plantings, additional soil excavation, backfill material, use of underdrains etc.), those landscaping expenses that would be required regardless of the bioretention installation should be subtracted when determining the net cost.

Perhaps of most importance, however, the cost savings compared to the use of traditional structural stormwater conveyance systems makes bioretention areas quite attractive financially. For example, the use of bioretention can decrease the cost required for constructing stormwater conveyance systems at a site. A medical office building in Maryland was able to reduce the amount of storm drain pipe that was needed from 240 to 70 m (800 to 230 ft) - a cost savings of $24,000 (PGDER, 1993). And a new residential development spent a total of approximately $100,000 using bioretention cells on each lot instead of nearly $400,000 for the traditional stormwater ponds that were originally planned. Also, in residential areas, stormwater management controls become a part of each property owner's landscape, reducing the public burden to maintain large centralized facilities.

*E1-2.8.8.2 Maintenance Costs*

The operation and maintenance costs for a bioretention facility will be comparable to those of typical landscaping required for a site. Costs beyond the normal landscaping fees will include the cost for testing the soils and may include costs for a sand bed and planting soil.

### E1-2.8.9 Research Needs

Bioretention is a relatively new, onsite stormwater treatment system; consequently, there are significant uncertainties about water quality performance and long term maintenance requirements to prevent clogging. Although there is little data on construction cost, this system consists mainly of landscaping and those costs can be estimated fairly easily.

### E1-2.8.10 References

Coffman, L.S., R. Goo and R. Frederick, 1999. "Low Impact Development: An Innovative Alternative Approach to Stormwater Management," Proceedings of the 26th Annual Water Resources Planning and Management Conference ASCE, June 6-9, Tempe, Arizona.

Davis, A.P., Shokouhian, M., Sharma, H., and Minani, C., 1998. *Optimization of Bioretention Design for Water Quality and Hydrologic Characteristics.*

Davis, A.P., M. Shokouhian, H. Sharma and C. Minami, 2001. "Laboratory Study of Biological Retention for Urban Stormwater Management," *Water Environment Research*, 73(1), 5-14.

Growing Greener in your Rappahannock River Watershed http://for.communitypoint.org/pages/grogreen.html

Hightshoe, G.L., 1988. Native Trees, Shrubs, and Vines for Urban and Rural America. Van Nostrand Reinhold, New York, New York.

Low Impact Development (LID) Urban Design Tools http://www.lid-stormwater.net/

Maryland Department of the Environment (MDE). 2000. *Maryland Stormwater Design Manual.* [http://www.mde.state.md.us/environment/wma/stormwatermanual].

Prince George's County Department of Environmental Resources (PGDER), 1993. Design Manual for Use of Bioretention in Storm water Management. Division of Environmental Management, Watershed Protection Branch. Landover, MD.

Prince George's County Department of Environmental Resources (PGDER), 1997. Bioretention Monitoring: Preliminary Data Analysis. Division of Environmental Management, Watershed Protection Branch. Landover, MD.

Prince George's County Department of Environmental Resources (PGDER), 2001. The Bioretention Manual. Programs and Planning Division, MD.

United States Environmental Pollution Agency Office of Water, 1999. Storm Water Technology Fact Sheet: Bioretention. EPA 832-F-99-012.

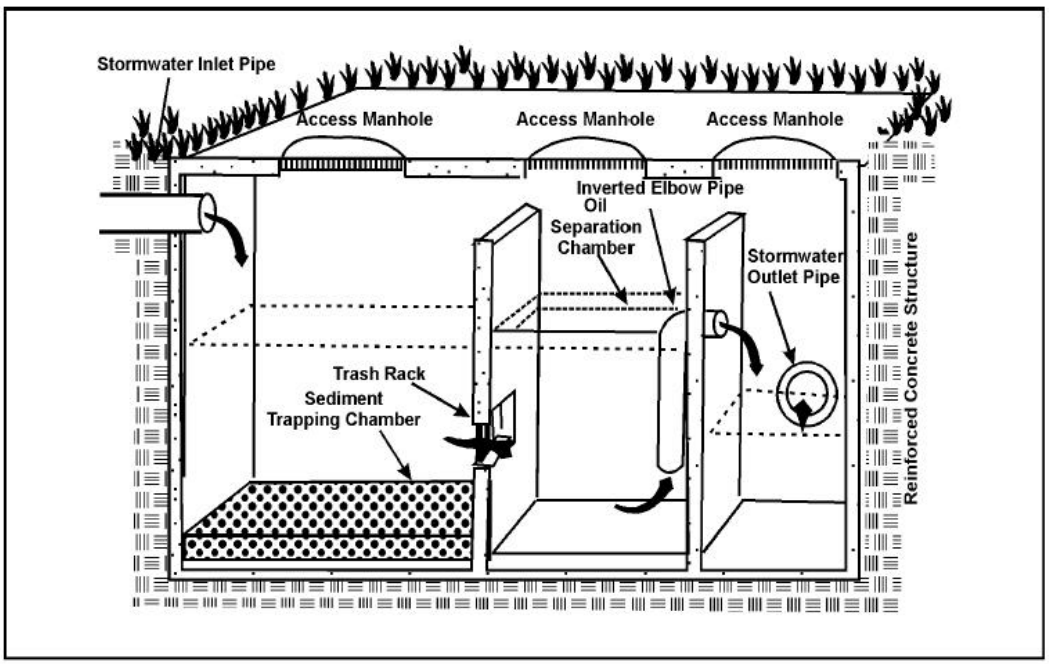
United States Environmental Protection Agency Office of Water, 2000. Low Impact Development (LID): A Literature Review. October 2000. EPA-841-B-00-005.

## E1-2.9 Water Quality Inlets/Oil Water Separators

### E1-2.9.1 Description

Water quality inlets (WQIs), also commonly called trapping catch basins, oil/grit separators or oil/water separators, consist of one or more chambers that promote sedimentation of coarse materials and separation of free oil (as opposed to emulsified or dissolved oil) from stormwater. Some WQIs also contain screens to help retain larger or floating debris, and many of the newer designs include a coalescing unit that helps promote oil/water separation. A typical WQI, as shown in Figure E1-28, consists of a sedimentation chamber, an oil separation chamber, and a discharge chamber. In the UK a simple version of a WQI, termed a gullypot, is often incorporated in urban and highway drainage systems, but has so little volume that substantial removal of pollutants is unlikely.

**Figure E1-28 Schematic of a Water Quality Inlet (Berg, 1991)**




### E1-2.9.2 Advantages

- Water quality inlets and oil/water separators are effective for capturing hydrocarbon spills.

### E1-2.9.3 Limitations

- WQIs generally provide limited hydraulic and residuals storage. Due to the limited storage, WQIs provide only marginal sediment removal and stormwater treatment.
- Standing water in the devices can provide a breeding ground for mosquitoes.
- Even ideally designed configurations cannot remove pollutants as well as other structural stormwater management practices, such as wet ponds, sand filters, and wet ponds.
- Unless frequently maintained, these devices can become a source of pollutants through resuspension.
- Typical concentrations of oil and grease in stormwater runoff may be below levels required for successful operation of these devices.

### E1-2.9.4 Design and Sizing Guidelines

- Water quality inlets are most effective for spill control and should be sized accordingly.

### E1-2.9.5 Performance

WQIs are most effective for collecting oil from spills, but also provide some removal of coarse sediment. A 1990 report by API found that the efficiency of oil and water separation in a WQI is inversely proportional to the ratio of the discharge rate to the unit's surface area. Due to the small capacity of the WQI, the discharge rate is typically very high and the detention time is very short. For example, the MWCOG study found that the average detention time in a WQI is less than 0.5 hour. This can result in minimal pollutant settling (API, 1990). However, the addition of coalescing units in many current WQI units may increase oil/water separation efficiency. Most coalescing units are designed to achieve a specific outlet concentration of oil and grease (for example, 10-15 mg/L oil and grease).

Grit and sediment are partially removed by gravity settling within the first two chambers. A WQI with a detention time of one hour may expect to have 20 to 40 percent removal of solids. Hydrocarbons associated with the accumulated sediments are also often removed from the runoff through this process. The WQI achieves slight, if any, removal of nutrients, metals and organic pollutants other than free petroleum products (Schueler, 1992).

The 1993 MWCOG study discussed above found that an average of less than 5 cm (2 in.) of sediments (mostly coarse-grained grit and organic matter) were trapped in the WQIs.

Hydrocarbon and total organic carbon (TOC) concentrations of the sediments averaged 8,150 and 53,900 milligrams per kilogram, respectively. The mean hydrocarbon concentration in the WQI water column was 10 mg/L. The study also indicated that sediment accumulation did not increase over time, suggesting that the sediments become re-suspended during storm events. The authors concluded that although the WQI effectively separates oil and grease from water, re-suspension of the settled matter appears to limit removal efficiencies. Actual removal only occurs when the residuals are removed from the WQI (Schueler 1992).

Pollutant removal in stormwater inlets can be somewhat improved using inserts, which are promoted for removal of oil and grease, trash, debris, and sediment. Some inserts are designed to drop directly into existing catch basins, while others may require extensive retrofit construction.

### E1-2.9.6 Siting Criteria

Oil/water separation units are often utilized in specific industrial areas, such as airport aprons, equipment washdown areas, or vehicle storage areas. In these instances, runoff from the area of concern will usually be diverted directly into the unit, while all other runoff is sent to the storm drain downstream from the oil/water separator. Oil/water separation tanks are often fitted with diffusion baffles at the inlets to prevent turbulent flow from entering the unit and re-suspending settled pollutants.

### E1-2.9.7 Design Guidelines

Prior to WQI design, the site should be evaluated to determine if another BMP would be more cost-effective in removing the pollutants of concern. WQIs should be used when no other BMP is feasible. The WQI should be constructed near a storm drain network so that flow can be easily diverted to the WQI for treatment (NVPDC, 1992). Any construction activities within the drainage area should be completed before installation of the WQI, and the drainage area should be revegetated so that the sediment loading to the WQI is minimized.

WQIs are most effective for small drainage areas. Drainage areas of 1 acre or less are often recommended. WQIs are typically used in an off-line configuration (i.e., portions of runoff are diverted to the WQI), but they can be used as on-line units (i.e., receive all runoff). Generally, off-line units are designed to handle the first 1.3 cm (0.5 in.) of runoff from the drainage areas. Upstream isolation/diversion structures can be used to divert the water to the off-line structure (Schueler, 1992). On-line units receive higher flows that will likely cause increased turbulence and resuspension of settled material, thereby reducing WQI performance.

Oil/water separation tanks are often fitted with diffusion baffles at the inlets to prevent turbulent flow from entering the unit and resuspending settled pollutants. WQIs are available as pre-manufactured units or can be cast in place. Reinforced concrete should be used to construct below-grade WQIs. The WQIs should be water tight to prevent possible ground water contamination.

#### E1-2.9.8 Maintenance

Frequent maintenance is required to prevent resuspension of accumulated sediment and displacement of captured oil and grease during subsequent storm events. Typical activities include trash removal if a screen or other debris capturing device is used, and removal of sediment using a vacuum (gully sucker or vactor truck) truck. Operators need to be properly trained in catch basin maintenance. Maintenance should include keeping a log of the amount of sediment collected and the date of removal. Some cities have incorporated the use of GIS systems to track sediment collection and to optimize future catch basin cleaning efforts.

One study (Pitt, 1985) concluded that catch basins can capture sediments up to approximately 60 percent of the sump volume. When sediment fills greater than 60 percent of their volume, catch basins reach steady state. Storm flows can then resuspend sediments trapped in the catch basin, and will bypass treatment. Frequent clean-out can retain the volume in the catch basin sump available for treatment of stormwater flows.

At a minimum, catch basins should be cleaned at least twice during the wet season. Two studies suggest that increasing the frequency of maintenance can improve the performance of catch basins, particularly in industrial or commercial areas. One study of 60 catch basins in Alameda County, California, found that increasing the maintenance frequency from once per year to twice per year could increase the total sediment removed by catch basins on an annual basis (Mineart and Singh, 1994). Annual sediment removed per inlet was 24 kg (54 lbs) for annual cleaning, 32 kg (70 lbs) for semi-annual and quarterly cleaning, and 73 kg (160 lbs) for monthly cleaning. For catch basins draining industrial uses, monthly cleaning increased total annual sediment collected to six times the amount collected by annual cleaning [82 kg (180 lbs) versus 4 kg (30 lbs)]. These results suggest that, at least for industrial uses, more frequent cleaning of catch basins may improve efficiency.

#### E1-2.9.9 Cost

A typical pre-cast catch basin costs between $2,000 and $3,000; however, oil/water separators can be much more expensive. The true pollutant removal cost associated with catch basins, however, is the long-term maintenance cost. A vacuum truck, the most common method of catch basin cleaning, costs between $125,000 and $150,000. This initial cost may be high for smaller Phase II communities. However, it may be possible to share a vacuum truck with another community. Typical trucks can store between 7.6 to 11 $m^3$ (10 and 15 yards) of material, which is enough storage for three to five catch basins. Assuming semi-annual cleaning, and that the truck could be filled and material disposed of twice in one day, one truck would be sufficient to clean between 750 and 1,000 catch basins. Another maintenance cost is the staff time needed to operate the truck. Depending on the regulations within a community, disposal costs of the sediment captured in catch basins may be significant.

E1-2.9.10 References

American Petroleum Institute (API), 1990. *Monographs on Refinery Environmental Control - Management of Water Discharges (Design and Operation of Oil-Water Separators).* Publication 421, 1st ed.

Aronson, G., D. Watson, and W. Pisaro. *Evaluation of Catch Basin Performance for Urban Stormwater Pollution Control.* U.S. Environmental Protection Agency, Washington, DC.

Berg, V.H., 1991. *Water Quality Inlets (Oil/Grit Separators)*. Maryland Department of the Environment, Sediment and Stormwater Administration.

Lager, J., W. Smith, R. Finn, and E. Finnemore. 1977. *Urban Stormwater Management and Technology: Update and Users' Guide.* Prepared for U.S. Environmental Protection Agency. EPA-600/8-77-014. 313 pp.

Metropolitan Washington Council of Governments (MWCOG), 1993. *The Quality of Trapped Sediments and Pool Water Within Oil Grit Separators in Suburban Maryland.* Interim Report.

Mineart, P., and S. Singh, 1994. *Storm Inlet Pilot Study*. Alameda County Urban Runoff Clean Water Program, Oakland, CA.

Northern Virginia Planning District Commission (NVPDC) and Engineers and Surveyors Institute, 1992. *Northern Virginia BMP Handbook.*

Pitt, R., and P. Bissonnette, 1984. *Bellevue Urban Runoff Program Summary Report*. U.S. Environmental Protection Agency, Water Planning Division, Washington, DC.

Pitt, R., M. Lilburn, S. Nix, S.R. Durrans, S. Burian, J. Voorhees, and J. Martinson, 2000. *Guidance Manual for Integrated Wet Weather Flow (WWF) Collection and Treatment Systems for Newly*

*Urbanized Areas (New WWF Systems)*. U.S. Environmental Protection Agency, Office of Research and Development, Cincinnati, OH.

Schueler, T.R., 1992. *A Current Assessment of Urban Best Management Practices*. Metropolitan Washington Council of Governments.

U.S. EPA, 1999. Stormwater Technology Fact Sheet: Water Quality Inlets, EPA 832-F-99-029, Office of Water, Washington DC.

## E1-2.10 Sand and Organic Filters

### E1-2.10.1 Description

Sand filters are usually two-chambered stormwater practices; the first is a settling chamber, and the second is a filter bed filled with sand or another filtering media. As stormwater flows into the first chamber, large particles settle out, and then finer particles and other pollutants are removed as stormwater flows through the filtering medium.

The use of slow-sand filters for the treatment of stormwater runoff is a fairly recent innovation. The filtration of water through sand as a means of improving its quality was first performed in 1829 in London to treat Thames River water. This first filter was the predecessor of the slow-sand type later developed in England and used extensively at the beginning of the 20th century in the United States for water and wastewater treatment. Over the intervening years, the use of slow-sand filters for the treatment of water and wastewater declined as improved rapid filtering and treatment technologies were developed (Young et al., 1996).

Since their original inception in Austin, Texas, hundreds of intermittent sand filters have been implemented to treat stormwater runoff. There have been numerous alterations or variations in the original design as engineers in other jurisdictions have improved and adapted the technology to meet their specific requirements. Major types include the Austin Sand Filter (Figure E1-29), the District of Columbia Underground Sand Filter, the Alexandria Dry Vault Sand Filter, the Delaware Sand Filter (Figure E1-30), and Multi-Chamber Treatment Train (Figure E1-31). The primary differences among these designs are location (i.e., above or below ground), the drainage area served, their filter surface areas, their land requirements, and the quantity of runoff they treat. Modifications to the traditional surface sand filter were made primarily to fit sand filters into more challenging design sites (e.g., underground and perimeter filters) or to improve pollutant removal (e.g., organic media filter).

The City of Austin recognizes two design alternatives: “full” and “partial” sedimentation. In the full sedimentation design the sedimentation basin is sized to contain the entire water quality volume, which is then released over 24 hours to the filter basin. The partial sedimentation design consists of a single basin, sized to contain the water quality volume, and is underlain by the filter bed. Because the amount of pretreatment in the partial sedimentation design is low, the City specifies a lower permeability for the sand, which results in a larger filter area as a safety factor.

**Figure E1-29 Schematic of the "Full Sedimentation" Austin Sand Filter**

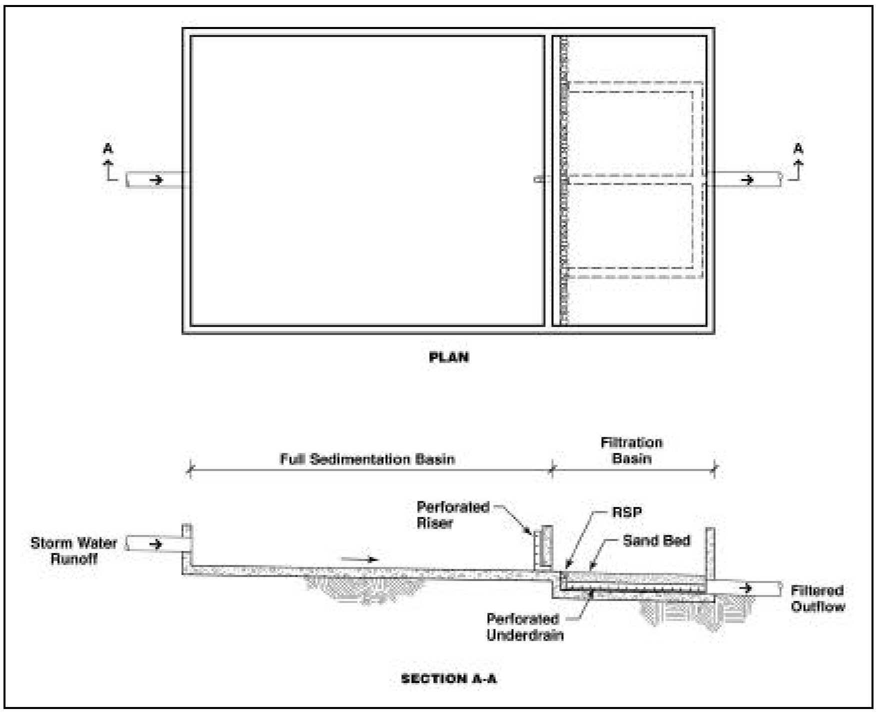


**Figure E1-30 Schematic of a Delaware Sand Filter (Young et al., 1996)**

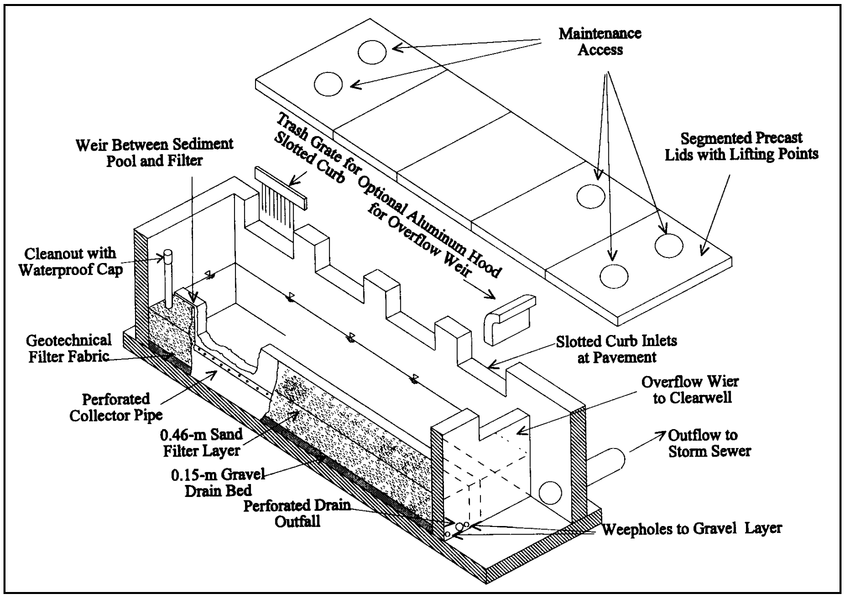
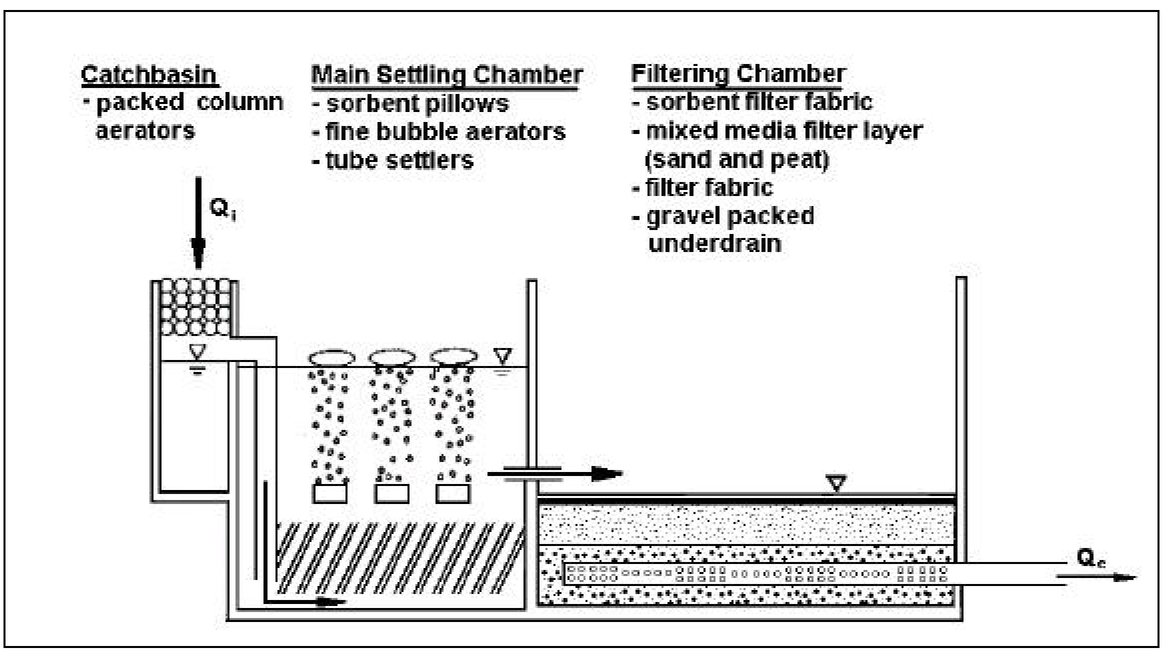



**Figure E1-31 Schematic of a Multi-Chambered Treatment Train**



### E1-2.10.2 Advantages

- Appropriate for space-limited areas
- Applicable in arid climates where wet basins and constructed wetlands are not appropriate
- High TSS Removal Efficiency

### E1-2.10.3 Limitations

- Require more maintenance that most other BMPs depending upon the sizing of the filter bed
- Generally require more hydraulic head to operate properly (minimum 3 feet)
- High solids loads will cause the filter to clog
- Work best for relatively small, impervious watersheds
- Filters in residential areas can present aesthetic and safety problems

### E1-2.10.4 Siting Criteria

In general, sand filters are preferred over infiltration practices, such as infiltration trenches, when contamination of groundwater with conventional pollutants - BOD, suspended solids, and fecal coliform - is of concern. This usually occurs in areas where underlying soils alone cannot treat runoff adequately - or ground water tables are high. In most cases, sand filters can be constructed with impermeable basin or chamber bottoms, which help to collect, treat, and release runoff to a storm drainage system or directly to surface water with no contact between contaminated runoff and groundwater. In regions where evaporation exceeds rainfall and a wet pond would be unlikely to maintain the required permanent pool, the Austin sand filtration system can be used.

The selection of a sand filter design depends largely on the drainage area's characteristics. For example, the Washington, D.C., and Delaware sand filter systems are well suited for highly impervious areas where land available for structural controls is limited, since both are installed underground. They are often used to treat runoff from parking lots, driveways, loading docks, service stations, garages, airport runways/taxiways, and storage yards. The Austin sand filtration system is more suited for large drainage areas that have both impervious and pervious surfaces. This system is located at grade and is often used at transportation facilities, in large parking areas, and in commercial developments. The Austin design normally occupies 1 to 4 percent of the drainage area.

It is challenging to use most sand filters in very flat terrain because they require a significant amount of hydraulic head [1.2 m (about 4 ft)], to allow flow through the system. One exception is the perimeter sand filter, which can be applied with as little as 0.6 m (2 ft) of head.

Sand filters are best applied on relatively small sites (up to 25 acres for surface sand filters and closer to 2 acres for perimeter or underground filters). Filters have been used on larger drainage areas, of up to 100 acres, but these systems can clog when they treat larger drainage areas unless adequate measures are provided to prevent clogging, such as a larger sedimentation chamber or more intensive regular maintenance.

When sand filters are designed as a stand-alone practice, they can be used on almost any soil because they can be designed so that stormwater never infiltrates into the soil or interacts with the ground water. Alternatively, sand filters can be designed as pretreatment for an infiltration practice, where soils do play a role.

#### E1-2.10.5 Design Guidelines

Pretreatment is a critical component of any stormwater management practice. In sand filters, pretreatment is achieved in the sedimentation chamber that precedes the filter bed. In this chamber, the coarsest particles settle out and thus do not reach the filter bed. Pretreatment reduces the maintenance burden of sand filters by reducing the potential of these sediments to clog the filter. Designers should provide at least 25 percent of the water quality volume in a dry or wet sedimentation chamber as pretreatment to the filter system. The water quality volume is the amount of runoff that will be treated for pollutant removal in the sedimentation chambers and filter bed.

Treatment design features help enhance the ability of a stormwater management practice to remove pollutants. In filtering systems, designers should provide at least 75 percent of the water quality volume in the sedimentation chambers and filter bed. In sand filters, designers should select a medium sand as the filtering medium. A fine aggregate (ASTM C-33) that is intended for use in concrete is commonly specified.

The filter bed should be sized using Darcy's Law, which relates the velocity of fluids to the hydraulic head and the coefficient of permeability of a medium. The resulting equation, as derived by the city of Austin, Texas, (1996), is

$$AF = WTV\ d / [k\ t\ (h+d)]$$

Where: WTV = Water Treatment Volume, $ft^3$

AF = area of the filter bed ($ft^2$);
d = depth of the filter bed (ft; usually about 1.5 feet, depending on the design);
k = coefficient of permeability of the filtering medium (ft/day);
t = time for the water quality volume to filter through the system (days; usually assumed to be 1.67 days); and
h = average water height above the sand bed (ft; assumed to be one-half of the maximum head).

Typical values for k, as assembled by CWP (1996), are shown in Table E1-33.

The permeability of sand shown in Table E1-33 is extremely conservative, but is widely used since it is incorporated in the design guidelines of the City of Austin. When the sand is initially installed, the permeability is so high that generally only a portion of the filter area is required to infiltrate the entire volume, especially in a "full sedimentation" Austin design where the capture volume is released to the filter basin over 24 hours.

**Table E1-33 Coefficient of Permeability Values for Stormwater Filtering Practices (CWP, 1996)**

| Filter Medium | Coefficient of Permeability (cm/day) | Coefficient of Permeability (ft/day) |
|---|---|---|
| Sand | 1.1 | 3.5 |
| Peat/Sand | 0.84 | 2.8 |
| Compost | 2.6 | 8.7 |

Typically, filtering practices are designed as "off-line" systems, meaning that during larger storms all runoff greater than the water quality volume is bypassed untreated using a flow splitter, which is a structure that directs larger flows to the storm drain system or to a stabilized channel. One exception is the perimeter filter; in this design, all flows enter the system, but larger flows overflow to an outlet chamber and are not treated.

All filtering practices, with the exception of exfiltration devices (see Design Variations) are designed with an under drain below the filtering bed. An under drain is a perforated pipe system in a gravel bed, installed on the bottom of filtering practices and used to collect and remove filtered runoff.

**Design Variations**

As mentioned earlier in this fact sheet, there are five basic stormwater filter designs--surface sand filter, underground filter, perimeter filter (also known as the "Delaware" filter), organic media filter, and Multi-Chamber Treatment Train. Other design variations can incorporate design features to recharge ground water or to meet the design challenges of cold or arid climates.

<u>Surface Sand Filter</u>

The surface sand filter is the original sand filter design. In this practice both the filter bed and the sediment chamber are aboveground. The surface sand filter is designed as an off-line practice, where only the water quality volume is directed to the filter. The surface sand filter is the least expensive filter option and has been the most widely used.

<u>Underground Sand Filter</u>

The underground sand filter is a modification of the surface sand filter, where all of the filter components are underground. Like the surface sand filter, this practice is an off-line system that receives only the smaller water quality events. Underground sand filters are expensive to construct but consume very little space. They are well suited to highly urbanized areas.

<u>Perimeter Sand Filter</u>

The perimeter sand filter also includes the basic design elements of a sediment chamber and a filter bed. In this design, however, flow enters the system through grates, usually at the edge of a parking lot. The perimeter sand filter is the only filtering option that is on-line, with all flows entering the system but larger events bypassing treatment by entering an overflow chamber. One major advantage to the perimeter sand filter design is that it requires little hydraulic head and thus is a good option in areas of low relief.

Organic Media Filter

Organic media filters are essentially the same as surface filters, with the sand medium replaced with or supplemented by another medium. Two examples are the peat/sand filter (Galli, 1990) and the compost filter system (CSF, 1996). The assumption is that these systems will have enhanced pollutant removal for many compounds because of the increased cation exchange capacity achieved by increasing the organic matter in the filter media.

Multi-Chamber Treatment Train

The Multi-Chamber Treatment Train (Robertson et al., 1995) is essentially a "deluxe sand filter." This underground system consists of three chambers. Stormwater enters into the first chamber, where screening occurs, trapping large sediments and releasing highly volatile materials. The second chamber provides settling of fine sediments and further removal of volatile compounds and also floatable hydrocarbons through the use of fine bubble diffusers and sorbent pads. The final chamber provides filtration by using a sand and peat mixed medium for reduction of the remaining pollutants. The top of the filter is covered by a filter fabric that evenly distributes the water volume and prevents channelization. Although this practice can achieve very high pollutant removal rates, it might be prohibitively expensive in many areas and has been implemented only on an experimental basis.

Exfiltration/Partial Exfiltration

In these designs, all or part of the under drain system is replaced with an open bottom that allows infiltration to the ground water. When the under drain is present, it is used as an overflow device in case the filter becomes clogged. These designs are best applied in the same soils where infiltration practices are used.

In cold climates, filters can be used, but surface or perimeter filters will not be effective during the winter months, and unintended consequences might result from a frozen filter bed. Using alternative conveyance measures such as a weir system between the sediment chamber and filter bed may avoid freezing associated with the traditional standpipe. Where possible, the filter bed should be below the frost line. Some filters, such as the peat/sand filter, should be shut down during the winter. These media will become completely impervious during freezing conditions. Using a larger underdrain system to encourage rapid draining during the winter months may prevent freezing of the filter bed.

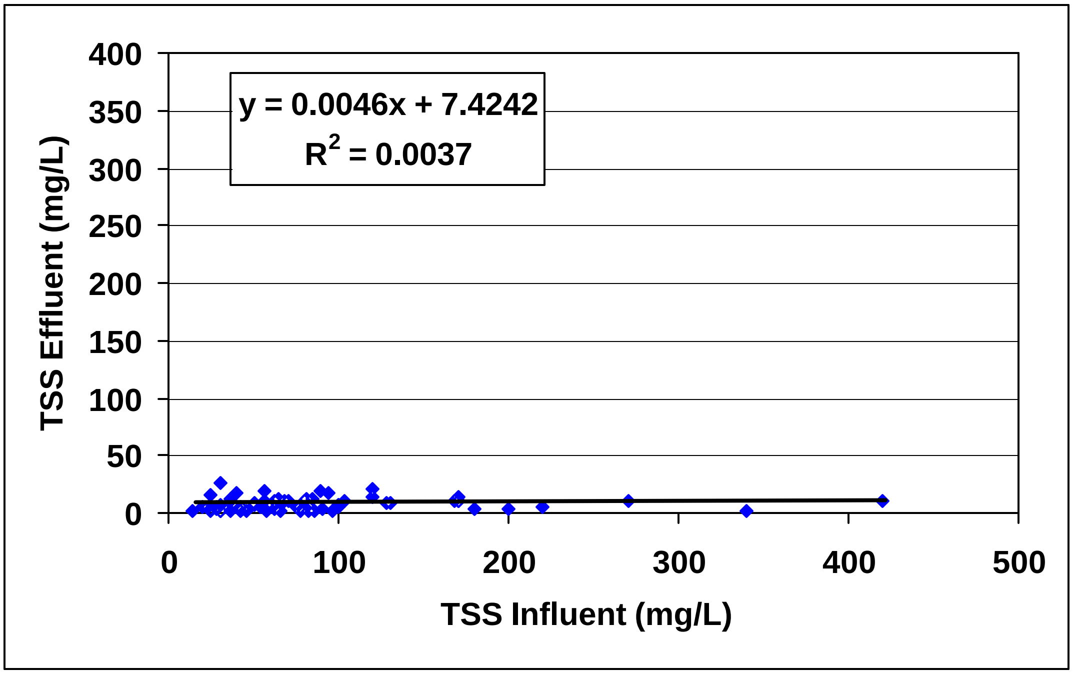
### E1-2.10.6 Performance

The pollutant removal performance of sand filters and other stormwater BMPs is generally characterized by the percent reduction in the influent load. This method implies a relationship between influent and effluent concentrations. For instance, it would be expected that a device that is reported to achieve a 75 percent reduction would have an effluent concentration equal to 25 percent of the influent concentrations. Recent work in California on Austin sand filter systems (sedimentation and filter basin) indicate that this model for characterizing performance is inadequate. Figure E1-32 presents a graph relating influent and effluent TSS concentrations for the Austin full sedimentation design.

It is clearly apparent that the effluent concentration is relative constant and independent of influent concentration. Consequently, the performance is more accurately characterized by the effluent concentration, which is about 7.5 mg/L. Constant effluent concentrations also are observed for all other particle-related constituents such as particulate metals (total - dissolved) and particulate phosphorus.

The small uncertainty in the estimate of the mean effluent concentration highlights the very consistent effluent quality for TSS produced by sand filters. In addition, it demonstrates that a calculated percent reduction for TSS and other constituents with similar behavior for Austin sand filters is a secondary characteristic of the device and depends primarily on the specific influent concentrations observed. The distinction between a constant effluent quality and a percent reduction is extremely important to recognize, if the results are to be used to estimate effluent quality from sand filters installed at other sites with different influent concentrations or for estimating compliance with water quality standards for storms with high concentrations of particulate constituents.

**Figure E1-32 Comparison of Influent and Effluent Concentrations for TSS**




If the conventionally derived removal efficiency (90 percent) were used to estimate the TSS concentrations in the treated runoff from storms with high influent concentrations, the estimated effluent concentration would be too high. For instance, the storm with the highest observed influent concentration (420 mg/L) would be expected to have a concentration in the treated runoff of 42 mg/L, rather than the 10 mg/L that was measured. In fact, the TSS effluent concentrations for all events with influent concentrations greater than 200 mg/L were 10 mg/L or less.

The stable effluent concentration of a sand filter under very different influent TSS concentrations implies something about the properties of the influent particle size distribution. If one assumes that only the smallest size fraction can pass through the filter, then the similarity in effluent concentrations suggests that there is little difference in the total mass of the smallest sized particles even when the total TSS concentration varies greatly. Further, the difference in TSS concentration must then be caused by changes in the relative amount of the larger size fractions. Further research is necessary to determine the range of particle size that is effectively removed in the filter and the portion of the size fraction of suspended solids that it represents in urban stormwater.

Sand filters are effective stormwater management practices for pollutant removal. Conventional removal rates for all sand filters and organic filters are presented in Table E1-34. With the exception of nitrates, which are always exported from filtering systems because of the conversion of ammonia and organic nitrogen to nitrate, they perform relatively well at removing pollutants.

**Table E1-34 Sand Filter Removal Efficiencies, %**

| | Sand Filter (Schueler, 1997) | Compost Filter System | | Multi-Chamber Treatment Train | | |
|---|---|---|---|---|---|---|
| | | Stewart, 1992 | Leif, 1999 | Pitt et al., 1997 | Pitt, 1996 | Greb et al., 1998 |
| TSS | 87 | 95 | 85 | 85 | 83 | 98 |
| TP | 51 | 41 | 4 | 80 | - | 84 |
| TN | 44 | - | - | - | - | - |
| Nitrate | -13 | -34 | -95 | - | 14 | - |
| Metals | 34-80 | 61-88 | 44-75 | 65-90 | 91-100 | 83-89 |
| Bacteria | 55 | - | - | - | - | - |

From the few studies available, it is difficult to determine if organic filters necessarily have higher removal efficiencies than sand filters. The Multi-Chamber Treatment Train may have high pollutant removal for some constituents, although an evaluation of these devices by the California Department of Transportation indicated no significant difference for most conventional pollutants.

### E1-2.10.7 Maintenance

Even though sand filters are generally thought of as one of the higher maintenance BMPs, in a recent California study an average of only about 49 hours a year was required for field activities. This was less maintenance than was required by extended detention basins serving comparable sized catchments. Most maintenance consists of routine removal of trash and debris, especially in Austin sand filters where the outlet riser from the sedimentation basin can become clogged.

Most data (i.e., Clark, 2001) indicate that hydraulic failure from clogging of the sand media occurs before pollutant breakthrough. Typically, only the very top of the sand becomes clogged while the rest remains in relative pristine condition as shown in Figure E1-33. The rate of clogging has been related to the TSS loading on the filter bed (Urbonas, 1999); however, the data are quite variable. Empirical observation of sites treating urban and highway runoff indicates that clogging of the filter occurs after 2 – 10 years of service. Presumably, this is related to differences in the type and amount of sediment in the catchment areas of the various installations. Once clogging occurs the top 2 – 3 inches of filter media is removed, which restores much, but not all, of the lost permeability. This removal of the surface layer can occur several times before the entire filter bed must be replaced. The cost of the removal of the surface layer is not prohibitive, generally ranging between $2,000 (EPA Fact Sheet) and $4,000 (Caltrans, 2002) depending on the size of the filter.

**Figure E1-33 Formation of Clogging Crust on Filter Bed**

Recommended maintenance activities and frequencies are listed in Table E1-35.

E1-2.10.8 Cost

*E1-2.10.8.1 Construction Costs*

There are few consistent published data on the cost of sand filters, largely because, with the exception of Austin, Texas, Alexandria, Virginia, and Washington, DC, they have not been widely used. Furthermore, filters have such varied designs that it is difficult to assign a cost to filters in general. A study by Brown and Schueler (1997) was unable to find a statistically valid relationship between the volume of water treated in a filter and the cost of the practice. The EPA filter fact sheet indicates a cost for an Austin sand filter at $18,500 (1997 dollars) for a 1 acre drainage area. However, the same design implemented at a site with a 0.45 acres. drainage area by the California Department of Transportation, cost $240,000. Consequently, there is a tremendous uncertainty about what the average construction cost might be.

**Table E1-35 Typical Maintenance Activities for Sedimentation Basin and Filter Bed**

| Activity | Schedule |
|---|---|
| • Inspect for standing water, sediment, trash and debris, and to identify potential problems. | Semiannually |
| • Remove accumulated trash and debris in the sedimentation basin, from the riser pipe, and the filter bed during routine inspections. | Semiannually |
| • Inspect the facility once during the wet season after a large rain event to determine whether the facility is draining completely within 72 hours. | Annually |
| • Remove top 50 mm (2 in.) of sand and dispose of sediment if facility drain time exceeds 72 hr. Restore media depth to 450 mm (18 in.) when overall media depth drops to 300 mm (12 in.). | As needed |
| Remove accumulated sediment in the sedimentation basin every 10 years or when the sediment occupies 10 percent of the basin volume, whichever is less. | Every 10 years or less. |

It is important to note that, although underground and perimeter sand filters can be more expensive than surface sand filters, they consume no surface space, making them a relatively cost-effective practice in ultra-urban areas where land is at a premium.

Given the number of facilities installed in the areas that promote their use it should be possible to develop fairly accurate construction cost numbers through a more comprehensive survey of municipalities and developers that have implemented these filters.

*E1-2.10.8.2 Maintenance Costs*

Annual costs for maintaining sand filter systems average about 5 percent of the initial construction cost (Schueler, 1992). Media is replaced as needed, with the frequency correlated with the solids loading on the filter bed. Currently the sand is being replaced in the DC filter systems about every 2 years, while an Austin design might last 3-10 years depending on the watershed characteristics. The cost to replace the gravel layer, filter fabric and top portion of the sand for DC sand filters is approximately $1,700 (1997 dollars).

Caltrans estimated future maintenance costs for the Austin design, assuming a device sized to treat runoff from approximately 1.6 ha (4 acres). These estimates are presented in Table E1-36 and assume a fully burdened hourly rate of $44 for labor. This estimate is somewhat uncertain, since complete replacement of the filter bed was not required during the period that maintenance costs were recorded.

**Table E1-36 Expected Annual Maintenance Costs for an Austin Sand Filter**

| Activity | Labor Hours | Equipment and Materials, $ | Cost, $ |
|---|---|---|---|
| Inspections | 4 | 0 | 176 |
| Maintenance | 36 | 125 | 1,706 |
| Vector Control | 0 | 0 | 0 |
| Administration | 3 | 0 | 132 |
| Direct Costs | - | 888 | 888 |
| **Total** | 43 | $1,013 | **$2,902** |

### E1-2.10.9 Research Needs

Sand filters have been widely used and monitored in a few areas of the US, so their siting criteria, design guidelines, and performance for most common constituents is well understood. One area related to performance that could be investigated is the use of media other than sand to improve pollutant removal for dissolved constituents such as nutrients and metals.

There is little documentation of actual maintenance activities and their associated costs. Additional work in this area is recommended.

Data reported on construction costs for these devices is highly variable, so some additional effort to identify the reasons for the wide ranges of costs reported is warranted.

### E1-2.10.10 References

AquaLogic™ Stormwater Filtration System Engineering Manual. AquaLogic Stormwater Abatement Filter Systems. San Antonio, TX. August 2000.

Barton Springs/Edwards Aquifer Conservation District, 1996. *Final Report: Enhanced Roadway Runoff Best Management Practices*. City of Austin, Drainage Utility, LCRA, TDOT. Austin, TX. 200 pp.

Bell, W., L. Stokes, L.J. Gavan, and T.N. Nguyen, 1995. *Assessment of the Pollutant Removal Efficiencies of Delaware Sand Filter BMPs*. Final Report. Department of Transportation and Environmental Services. Alexandria, VA. 140 pp. Also in Performance of Delaware Sand Filter Assessed. *Watershed Protection Techniques*.

Center for Watershed Protection. Fall 1995. Vol. 2(1): 291–293.

Brown, W., and T. Schueler, 1997. *The Economics of Stormwater BMPs in the Mid-Atlantic Region*. Prepared for the Chesapeake Research Consortium, Edgewater, MD, by the Center for Watershed Protection, Ellicott City, MD.

Center for Watershed Protection (CWP)., 1996. *Design of Stormwater Filtering Systems*. Prepared for the Chesapeake Research Consortium, Solomons, MD, and U.S. EPA Region 5, Chicago, IL, by the Center for Watershed Protection, Ellicott City, MD.

Center for Watershed Protection (CWP), 1997. Multi-Chamber Treatment Train developed for stormwater hot spots. *Watershed Protection Techniques* 2(3):445–449.

City of Austin, TX, 1990. *Removal Efficiencies of Stormwater Control Structures*. Final Report. Environmental Resource Management Division. 36 p. Also in: Developments in Sand Filter Technology to Improve Stormwater Runoff Quality. *Watershed Protection Techniques*. Center for Watershed Protection. Summer 1994. Vol. 1(2): 47–54.

City of Austin, TX, 1996. *Design of Water Quality Controls*. City of Austin, TX.

Clark, S.E., 2000, Urban Stormwater Filtration: Optimization of Design Parameters and a Pilot-Scale Evaluation, Ph.D. Dissertation, University of Alabama at Birmingham.

CSF Treatment Systems, Inc. (CSF), 1996. *Stormwater Management Promotional Brochure*. CSF Treatment Systems, Inc., Portland, OR.

Curran, T., 1996. Peat Sand Efficiency Calculations for McGregor Park. Unpublished data. Lower Colorado River Authority. Austin, TX.

Galli, F., 1990. *Peat-Sand Filters: A Proposed Stormwater Management Practice for Urban Areas*. Metropolitan Washington Council of Governments, Washington, DC.

Glick, Roger, Chang, George C., and Barrett, Michael E., 1998. Monitoring and evaluation of stormwater quality control basins, in *Watershed Management: Moving from Theory to Implementation*, Denver, CO, May 3-6, 1998, pp 369 – 376.

Greb, S., S. Corsi, and R. Waschbush, 1998. Evaluation of Stormceptor© and Multi-Chamber Treatment Train as Urban Retrofit Strategies. Presented at Retrofit Opportunities for Water Resource Protection in Urban Environments, A National Conference. The Westin Hotel, Chicago, IL, February 10–12, 1998.

Harper, H., and J. Herr, 1993. *Treatment Efficiency of Detention With Filtration Systems*. Environmental Research and Design, Inc. Final Report Submitted to Florida Department of Environmental Regulation. Orlando, FL. 164 pp.

Horner, R.R. and Horner, C.R., 1999. "Performance of a Perimeter ("Delaware") Sand Filter in Treating Stormwater Runoff from a Barge Loading Terminal," in *Proceedings of the Comprehensive Stormwater and Aquatic Ecosystem Management Conf.*, Auckland, N.Z., Feb. 1999, pp 183-192.

Horner, R.R., and C.R. Horner, 1995. *Design, Construction and Evaluation of a Sand Filter Stormwater Treatment System*. Part II. *Performance Monitoring*. Report to Alaska Marine Lines, Seattle, WA. 38 p. Also in Performance of Delaware Sand Filter Assessed. *Watershed Protection Techniques*. Center for Watershed Protection. Fall 1995. vol. 2(1): 291–293.

Keblin, Michael V., Barrett, Michael E., Malina, Joseph F., Jr., Charbeneau, Randall J., 1998. *The Effectiveness of Permanent Highway Runoff Controls: Sedimentation/Filtration Systems*, Research Report 2954-1, Center for Transportation Research, University of Texas at Austin.

King County, Washington, Department of Natural Resources, 2000. *King County Surface Water Design Manual*. [http://splash.metrokc.gov/wlr/dss/manual.htm]. Last updated March 6, 2000. Accessed January 5, 2001.

Leif, T., 1999. *Compost Stormwater Filter Evaluation*. Snohomish County, Washington, Department of Public Works, Everett, WA.

Maryland Department of the Environment (MDE), 2000. *Maryland Stormwater Design Manual*. [http://www.mde.state.md.us/environment/wma/stormwatermanual]. Accessed May 22, 2001.

Pitt, R., 1996. "The Control of Toxicants at Critical Source Areas." Presented at the ASCE/Engineering Foundation Conference, Snowbird, UT, August 1996.

Pitt, R., M. Lilburn, and S. Burian, 1997. *Storm Drainage Design for the Future: Summary of Current U.S. EPA Research.* American Society of Civil Engineers Technical Conference, Gulf Shores, AL, July 1997.

Robertson, B., R. Pitt, A. Ayyoubi, and R. Field, 1995. "A Multi-Chambered Stormwater Treatment Train," In *Proceedings of the Engineering Foundation Conference: Stormwater NPDES-Related Monitoring Needs, Mt. Crested Butte, Colorado, August 7–12, 1994*, American Society of Civil Engineers, New York, New York.

Schueler, T., 1994. "Developments in Sand Filter Technology to Improve Stormwater Runoff Quality, *Watershed Protection Techniques* 1(2):47–54.

Schueler, T., 1997. "Comparative Pollutant Removal Capability of Urban BMPs: A Reanalysis," *Watershed Protection Techniques* 2(4):515–520.

Stewart, W., 1992. *Compost Stormwater Treatment System.* W&H Pacific Consultants. Draft Report. Portland, OR. Also in Innovative Leaf Compost System Used to Filter Runoff at Small Sites in the Northwest. *Watershed Protection Techniques*. Center for Watershed Protection. February 1994. Vol. 1(1): 13–14.

Urbonas, B.R., 1999, "Design of a Sand Filter for Stormwater Quality Enhancement," *Water Environment Research,* Vol. 71, No. 1, pp 102-113.

Washington State Department of Ecology (DOE), 1992. *Stormwater Management Manual for the Puget Sound Basin,* Washington State Department of Ecology, Olympia, WA.

Watershed Management Institute (WMI), 1997. *Operation, Maintenance, and Management of Stormwater Management Systems.* Prepared for U.S. EPA Office of Water, Washington, DC, by Watershed Management Institute.

Welborn, C., and J. Veenhuis, 1987. *Effects of Runoff Controls on the Quantity and Quality of Urban Runoff in Two Locations in Austin, TX.* USGS Water Resources Investigations Report. 87–4004. 88 pp.

Young, G.K., et al., 1996. *Evaluation and Management of Highway Runoff Water Quality*, Publication No. FHWA-PD-96-032, U.S. Department of Transportation, Federal Highway Administration, Office of Environment and Planning.

**Information Resources**

Center for Watershed Protection (CWP), 1997. *Stormwater BMP Design Supplement for Cold Climates*. Prepared for U.S. EPA Office of Wetlands, Oceans and Watersheds, Washington, DC, by the Center for Watershed Protection, Ellicott City, MD.

City of Alexandria, VA. Unconventional BMP Design Criteria. In *Alexandria Supplement to the Northern Virginia BMP Handbook*. Alexandria, VA.

Shaver, E. and R. Baldwin, 1991. *Sand Filter Design for Water Quality Treatment*. Delaware Department of Natural Resources and Environmental Control, Dover, DE.

## E1-2.11 Proprietary End-of-Pipe Controls

### E1-2.11.1 Introduction

The purpose of this section is to summarize current information for certain end-of-pipe proprietary products. The types of information summarized include:

- Engineering information such as flow capacities, dimensions, weight, and materials.
- Data on pollutant removal efficiency
- Maintenance procedures and frequencies.
- Capital and maintenance costs

The following products are reviewed:

1. Actiflow © (http://www.usfilter.com/water/ProductDescription.asp?WID=25&PID=252
2. BaySaver™ (http://www.baysaver.com/baysaver.html)
3. CDS™ (http://www.cdstech.com.au/us/index.htm)
4. Downstream Defender™ (http://hyn7657s1.verdi.2day.com/Hynds_env/dstreamdef.html)
5. Stormceptor® (http://www.stormceptor.com/index.php)

6. StormFilter® (http://www.stormwatermgt.com/products/stormfilter.shtml)
7. StormTreat™ (http://www.stormtreat.com/home.htm)
8. Vortechs™ (http://www.vortechnics.com/)
9. V2B1™ (http://www.env21.com/)

A brief description of each technology, its manufacturer, and the pollutants that are potentially removable are presented in Table E1-37. The installation history of each product is presented in Table E1-38. The total number of installations in the United States and Canada, California, Caltrans, and other state DOTs are shown.

The Actiflow© product is excluded from further discussion due to the complexity of the system, which is similar to wastewater treatment (coagulation/flocculation). It requires daily attention when operating and is more appropriate for treating CSO than stormwater. It is therefore not appropriate for treating discharges from individual municipal outfalls. As the Actiflow© system may have some use in large, regional stormwater treatment projects in which the County or others might participate, some information is included in Table E1-37.

E1-2.11.2 Advantages

- Small footprint and below grade installation for retrofit situations
- Can be highly effective for trash and debris
- Can serve a larger drainage area than inlet inserts

E1-2.11.3 Limitations

- Often less effective for removal of conventional stormwater pollutants
- Proprietary products may be difficult for many agencies to specify
- May be locked into a sole source for replacement material
- No reduction of peak flows or runoff volume

## Table E1-37 Summary of Product Descriptions

| System | System Description | Pollutants Potentially Removed[1] | Pollutant Removal Mechanisms |
|---|---|---|---|
| Actiflow© | Stormwater treated with coagulant and microsand that enhances settling. Mixture settles in clarifier. Sand is separated from settled sediment with hydrocyclone and reused. May be more appropriate for CSOs. | Particulate pollutants | Coagulation, flocculation, settling. |
| BaySaver™ | Consists of two standard manholes. The first is for removal of sediment and separation of floatables, which are diverted by a special device into the second manhole for storage. Diversion device also passes extreme flows to bypass through the unit. Three flow sizes, 12 models. | Floatables including oil/grease and particulate pollutants. | Gravity settling, bypass to prevent resuspension of sediments, protection of floatables movement to a second manhole, and . |
| CDS™ | Circular device; flow is directed to create circular flow like a vortex, but removal occurs as the water passes through a screen around the outer perimeter. Removal induced by countercurrent flows on opposite sides of the screen, which also prevents clogging of the screen. 11 flow sizes. | Floatables, and particulate pollutants | Trapping of floatables and sediments by differential velocities created by a countercurrent flow next to a screen |
| Downstream Defender™ | Uses swirl-flow concentrator concept with device installed in a round single manhole vault. Four flow sizes. | Floatables and particulate pollutants | Swirl technology to enhance removal of sediments in small space. |
| Stormceptor® | A weir insert is placed in a round manhole vault to improve hydra ulics thereby improving removal efficiency and retention of sediment. During low flows the insert directs the flow downward and then laterally towards the walls of the sump. Above the treatment flow rate, the excess flow above the design flow rate flows directly across the insert device towards the outlet. Three flow sizes in eight models. | Floatables including oil/grease and particulate pollutants | Gravity settling enhanced by improved hydraulic conditions in what is essentially a large manhole. |
| StormFilter® | Vertical cylinder with media of various types placed in the cylinder. Water enters laterally through the filter, enters a vertical center well which exits to an underdrain system. One standard size cylinder (15 gpm). Number of cylinders is a function of design peak flow. Pretreatment desirable under circumstances as defined by the manufacturer. | Varies with the media. All reduces particulate pollutants. CSF™ (compost) also reduces dissolved metals. Zeolite removes dissolved phosphorus. | Gravity settling, filtration of TSS by filtration media, and with some media, removal of dissolved metals or phosphorus by adsorption or ion exchange on the media. |
| StormTreat™ | Circular device consisting of two circular chambers, closed inner chamber for settleables and floatables, and open outer chamber with wetland vegetation that is supported in gravel. One size, about 9.5 diameter, off-line unit with live volume of 1400 gallons. Fills each storm, slowly drains after storm in 5 to 10 days. Several units placed together with flow manifold to match design flow. Pretreat to remove gross solids is required. | Particulate and dissolved pollutants, oil/grease, and bacteria. | Gravity settling, filtration of TSS by screens and soil that supports wetland vegetation that also removes dissolved metals and phosphorus by adsorption or ion exchange. Nutrient uptake by the vegetation. |
| Vortechs™ | Swirl-flow concentration with the swirl device placed in a rectangular, shallow vault. Comes in nine standard sizes. | Floatables including oil/grease and particulate pollutants | Swirl technology to enhance removal of sediments in small space. |
| V2B1™ | Two manholes in series. Swirl flow concentration removes particulates and floatables in first manhole. Floatables move to chamber in second manhole for storage. Diverter in first manhole bypasses high flows from first to second manhole. Seven models. | Floatables including oil/grease and particulates | |

1. As stated by each manufacturer.

**Table E1-38 Installation Status**

| System | Total | Caltrans | California | Other DOTs | Other Road Systems |
|---|---|---|---|---|---|
| BaySaver | 24[2] | 0 | 0 | 0 | 0 |
| CDS | 160[3] | 0 | 7 | 0 | 4 |
| Downstream Defender | 40 | 0 | 0 | yes[4] New Hampshire, Washington | yes[4] New Hampshire, Washington |
| Stormceptor | 2000 ± | 0[5] | 56 | 24 Ohio, Connecticut, Washington Texas, Oregon, Massachusetts | no data |
| StormFilter | 300 | 42[6] | 45[6] | 3 | 0 |
| StormTreat | 105 | 0 | 13 | yes[4] Massachusetts, New Hampshire, | yes[4] |
| V2B1 | 59 | 0 | 0 | | |
| Vortechs | 500 ± | 0 | 3 | 17 Connecticut, Maine, Minnesota, New Hampshire, New Jersey, New York, Washington | yes[4] |

1. Anticipate 300 to 400 installations in 1999.
2. Additional 40 under design.
3. 120 units in Australia, 20 units in U.S.
4. Numbers not provided.
5. 12 units will be installed soon in District 3.
6. Includes the 41 units installed in the San Joaquin Hills Transportation Corridor of which all are the older flat bed technology.

E1-2.11.4 Siting and Operational Considerations

End-of-pipe systems are designed for use in space-constrained installations and consequently have been heavily promoted for implementation within highway rights-of-way and in ultra-urban settings. Some of the smaller systems have footprints of less than 5 $m^2$ (50 $ft^2$). Many of the systems are completely enclosed and may be placed under sidewalks or in other areas where the public has access.

Accumulation of litter and debris in these systems can be very rapid and greatly reduce the capacity of the devices. Consequently, they must be installed at sites with easy access for maintenance crews and their equipment. Required equipment often includes vactor trucks for removing accumulated sediment and runoff.

The maximum flow that can be treated by each type of unit varies and this limits the maximum drainage area for each device. Some of the units also require greater head to operate effectively, which limits their potential implementation in some retrofit situations. A summary of information for key engineering attributes is presented in Table E1-39. These include whether this system is surface or subsurface, treatment flow capacities, hydraulic flow capacities, general configuration, space requirements, weight, and head loss. Table E1-39 indicates that the products varied widely with respect to basic engineering attributes.

Treatment flow capacity is the peak rate to which the unit treats stormwater. The hydraulic capacity is the maximum flow that can be passed through the unit. A bypass external to the product is needed if flows are expected to exceed the capacity indicated. The number of models with different flow capacities for each product varies from three to eleven. The upper capacity of each product also varies considerably, from 2.5 cfs (Stormceptor©) to 300 cfs (CDS™). StormFilter™ and StormTreat™ both consist of one standard unit that is assembled in appropriate multiples to fit the needs of the site.

All except the StormTreat™ are sized according to the peak flow of the design event. For StormTreat, an off-line system, stormwater detention must also be provided to reduce the runoff rate. The number of units is determined by the volume of water that must be treated and the desired time to drain the detention facility.

The capacities identified in Table E1-39 are derived from engineering calculations. However, the flow regime of some of the products is rather complicated with internal bypasses and diversions. Hence, the stated capacity as well as internal diversion rates where appropriate

## Table E1-39 Summary of Engineering Information on Each Product[1]

| System | Surface Or Subsurface | Treatment Flow Capacities | Hydraulic Flow Capacities | General Configuration | Space Used (+)[2] Dia X D Or W X L X D | Weight[3] | Head Loss[4] |
|---|---|---|---|---|---|---|---|
| **BAYSAVER** ä | Subsurface | 2.4, 7.2, and 11.1 cfs. | 8.8, 24.5 and 39.6 cfs | Two circular manholes, 4' to 6' diameters | Min: 10 x 14 x 4<br>Max: 13 x 18 x 8 | Weight of manhole | Information not provided. |
| CDS™ | Subsurface | 1.1 to 300 cfs 11 models | Of the storm drain | Circular | Min: 4.5' x 5.5'<br>Max: 41'x varies[5] | Min: < 500<br>Max: 43,460 | Min: 0.31'<br>Max: 2.60' |
| Downstream Defender™ | Subsurface | 0.75, 3, 7, and 13 cfs. | 3, 8, 15, and 25 cfs | One circular manhole vault, 4' to 10' diameters | Min: 6' x 8'<br>Max: 12' x 15' | Min: 13,200<br>Max: 140,300 | Min: 23"<br>Max: 33" |
| Stormceptor® | Subsurface | 0.65, 1.1, 1.8 and 2.5 cfs. | Available height over weir and the storm drain | One circular manhole vault, 6' to 12' diameters | Min: 8.5' x 6.5'<br>Max: 14' x 15'[6] | Min: 5,860<br>Max: 27,840 | 1" between inlet and outlet inverts |
| StormFilter® | Either | 0.03 cfs and up | Precast: 2.2 cfs<br>Linear: 1.3 cfs<br>Cast in place: 2 cfs per 4' of weir | Cartridges placed in rectangular vaults | Precast Min: 7'x 9'x 5'(4)<br>Max: 10'x 44'x 12' (56)<br>Lineal Min: 3' x 10' 3' (3)<br>Max: 3 x 20' x? (8)<br>Cast Min: 10'x 40'x 5'<br>Max: open<br>(number of cartridges) | Weight of vault | Min: 2.3'<br>Max: NA |
| StormTreat™ | Surface | 1,400 gallons/unit | NA (offline) | Circular | Min: 9.5' x 4'<br>Max: NA | Min: 300<br>Max: NA | Head required to fill unit is minimal |
| Vortechs™ | Subsurface | 1.6 to 25 cfs nine models | 1.6 to 25 cfs[7] | Rectangular vault | Min: 3' x 9' x 8' of 9.5'<br>Max: 12' x 18' x 8' or 9.5' | Min: 27,000<br>Max: 47,000 | Min: 4"<br>Max: 19" |
| V2B1™ | Subsurface | 2.8 to 25.2 cfs seven models | 2.8 to 25.2 cfs | Two circular manhole vaults, 4' to 12' diameter | Min: 6' x 12'x 7'<br>Max:14'x 27'x 11' | Min:<br>Max: | Typically 10" to 18" |

1. Some of the enclosed information provided by personal communication and is not present in the manufacturer's brochure material.
2. For subsurface product, excludes riser to surface, which is variable, excludes ballast.
3. Of the heaviest component of each model delivered to the site, which in many cases is the entire unit preassembled. Excludes ballast. Excludes cast in place models.
4. Range of maximum heads required by the various models of each product.
5. Variable depth for the 3 largest units which are cast in place.
6. Assuming 24" pipe.
7. This is the capacity at 100 gpm/ft2. However, each model is able to pass a somewhat higher, unstated, flow as the water is able to rise an additional 3 inches gap over the top of the flow control wall.

should be confirmed with hydraulic tests. These tests should be done with smaller units, the results of which can be scaled upward to larger models that are impractical to test given the constraints of water availability in a hydraulics lab.

The information on weights in Table E1-39 identifies the weight of the largest component delivered to the site which, in the case of the smallest models, is typically the entire unit. The products vary widely with regard to the elevation drop that is required to properly function hydraulically.

### E1-2.11.5 Performance

Table E1-40 provides a summary of the studies that have been conducted to date and indicates whether the studies conform to minimum protocols adopted by Caltrans for evaluating stormwater BMPs. The parameter typically evaluated by either the manufacturers or the purchasers of the proprietary products is TSS. Few studies have included other pollutants. The only product for which an extensive array of pollutant removal efficiencies has been evaluated is the StormFilter™ with leaf compost (CSF™). However, most of these data are of the evaluation of the older flat bed configuration, which has been replaced by the cartridge system and a new filter media comprised only partially of leaf compost.

Most of the studies have been very limited in scope. A common deficiency has been the sampling protocol. Few studies have evaluated flow-weighted composite samples gathered through each storm. Most studies have taken only single paired grabs of the influent and effluent during each storm, or multiple time-paced grab samples. The value of single paired samples is very problematic, particularly for those products that retain water between storms. With these products one effluent sample may actually represent conditions prior to the storm being sampled.

#### BaySaver

The BaySaver system is essentially untested at this time. One performance study has been conducted. Three storms were sampled, but only one of which was of notable size. The average rate of flow into the unit for the three storms ranged from only 4% to 6% of the test unit's capacity. During one storm the peak inflow reached about 50% of capacity, but it is not clear if sampling occurred during this peak period.

CDS

The CDS™ system is likely the most effective of all of the proprietary end-of-pipe technologies. CDS™ has shown that its product is highly efficient at containing gross, floatable solids. It is capable of removing sediments with diameters considerably less than the size of the screen. However, a screen at least as small as 600 microns, and possibly 400 microns, is necessary to obtain high removal efficiencies for particles in the range of 50 to 150 microns. Unfortunately, the number of constituents that have been monitored is very small. For instance, there is no data for removal of nutrients or metals. Caltrans plans to monitor two of these devices as part of the BMP Retrofit Study; consequently, performance data from Southern California will be available within a couple of years.

Downstream Defender™

This system has not been field-tested. The system has been evaluated in the laboratory by adding sediment to tap water. However, the size distribution of the material used in this test was not representative of the distribution of sediment in stormwater.

Stormceptor®

While several studies have been conducted on removal efficiency and characteristics of the sediment, only the study by Grebs et al. (1998) of the Wisconsin Department of Natural Resources meets the Caltrans criteria of multiple storms and flow-weight composite samples. Based on Stormceptor's Technical Manual the efficiency of the unit tested by Greb et al. (1998) should have been in excess of 80%. However, the efficiency was only 26%. At issue however was the a situation that could have biased the results; high salt concentrations from stored road sand created a zone of very high density water in the lower half of the sump. This could have created a short circuiting situation. However, the efficiency of TSS removal for first 14 storms, taken before salt buildup was an issue, was still only 31%.

StormFilter

StormFilter is the only product that has been evaluated using flow-weighted composite samples. Monitoring of the older flat bed design indicates fairly good removal of most stormwater constituents except for nutrients. Tests at the University of Texas of the compost media confirm these results. This design has been replaced with a cartridge filter design and the single study conducted on this design indicates very poor performance, with negative removals for TSS and many other constituents. Additional monitoring of this design at the Caltrans Kearny Mesa Maintenance Yard will help evaluate the performance of the new design.

StormTreat™

While one study indicates very high removal efficiencies, the samples were not flow weight composites. This inadequacy is perhaps of less concern with respect to the effluent as the StormTreat releases water slowly, requiring a relatively large number of units to treat modest tributary areas. Based on a Caltrans design storm, it has been estimated that about five units are required per impervious acre treated.

Vortechs™

The one study of this system sampled seven storms, but the samples were not flow-weighted composite.

V2B1™

This system is untested at this time. Only lab tests with sediment altered tap water have been conducted.

E1-2.11.6 Maintenance

Table E1-41 contains a summary of maintenance information provided by each manufacturer including the storage capacities for sediment and floatables. The total capacity can be determined by the use of the data provided in the last column. As indicated in the table, most manufacturers recommend either quarterly or annual maintenance; however, this is not likely to be sufficient given the amount of litter and other debris which can clog inlets, filters and other parts of these devices. Given the limited field performance data for most of these devices, inspections should be scheduled after each storm during the first wet season after installation to ensure adequate performance. Quarterly inspections are probably sufficient for the dry season. After the first year, inspection frequency can be reduced if experience warrants.

In addition to inspections and debris removal, other maintenance items include sediment removal (which requires that all runoff in the device be pumped out), pumping to remove free phase hydrocarbons, and replacement of filter media (StormFilter).

Table E1-41 indicates that the sediment capacity differs between the products, as well as between the models of each product. That a particular product has a lower capacity does not necessarily mean the design is inferior, as the smaller capacities may be sufficient for the needs of the

## Table E1-40 Summary of Available Data on Performance

| System | Tests Of Treatment Performance | | | | Did Tests Meet Caltrans Protocol |
|---|---|---|---|---|---|
| | # of studies/sites | type[1] | Constituents[2] | Comments | |
| BaySaver™ | 1/1 | F | TSS | Very small storms in comparison to the capacity of the unit tested. Value of the field study limited by the type of sampling. Other analytes were evaluated but manufacturer does not consider the data valid. | No, too few storms (3) and not flow-weight composites. |
| CDS™ | 5/5 | F, L | TSS, litter, oil/grease, characteristics of captured sediment and litter. | Studies focused on characteristics of litter and sediments not efficiency. Further, the studies are of units with screens much larger than would be appropriate for the US, limiting the value of the findings with regard to sediment capture and pollutant reduction. See text. | NA. Tests not done with small screens except one lab study that was done at constant flow. |
| Downstream Defender™ | 2/2 | F, L | TSS, characteristics of captured sediment | Studies of TSS efficiency limited to lab tests; size distribution used in lab test was not representative of stormwater. See text. | No. No field studies |
| Stormceptor ® | 10/29 | F, L, C | TSS, BOD, COD, TOC, oil/grease, cadmium, copper, zinc, lead, characteristics of captured sediment, phosphorus, nitrogen forms, | Majority of data is of size distribution of captured sediment. Performance evaluated at 5 sites. | One study meets the Caltrans protocol: Greb et al., 1998. |
| StormFilter® | 8/9 | F, L | TSS, COD, TDS, TOC, DOC, oil/grease, TPH, PAH, total and dissolved metals (chromium, copper, lead, nickel, zinc), total metals (aluminum) phosphorus and dissolved phosphorus, nitrogen forms, fecal coliform, characteristics of captured sediment | Most of the data are the old flat bed system. Four of the five studies of the cartridge were single grabs only, limiting the value of those studies. | SMI study of its original prototype and the study of the North Hollywood and Bonita Canyon units meets the protocol |
| StormTreat™ | 1 | F | TSS, TPH, COD, nitrogen (total), phosphorus, TP, fecal coliform, zinc | Too few influent samples were taken in each storm limiting the value of the efficiency estimates. | No, too few storms and not flow-weight composites. |
| Vortechs™ | 4 | F, L, C | TSS | Value of the two field studies limited by type of sampling and other issues. One study evaluated a much more extensive list of analytes. However, the author cautions on the use of this data. | One study did flow-weight composites: USA, 1998. |
| V2B1™ | 1 | L | TSS | | No field studies conducted. |

1. F = field; L = lab; C = computer simulation
2. Constituents analyzed but found to be below the detection limit in the influent are not included.

## Table E1-41 Summary of Maintenance Information for Each Product[1]

| System | Treatment Flow Capacity Ranges | Manufacturers Recommendations | Storage Capacities[1] Sediment $ft^3$[2] | Floatables gallons | Storage/CFS SE Sediment $ft^3/cfs$[1,3] | Flotables gallons/cfs | Cleanout at % of Sump Capacity[4] |
|---|---|---|---|---|---|---|---|
| BaySaver™ | 2.4 to 11.1 cfs | Frequency not specified | 50 to 200 $ft^3$[3,5] | 384 to 868 | 10 to 33 | 78 to 160 | 0.25 |
| CDS™ | 1.1 to 300 cfs. | Clean sump seasonally four times per year and inspect screen annually. Clean sump when 85% full or if floatables exceeds 2' thickness | 16 to XXX $ft^3$ varies on units >148 cfs which are cast in place | 105 to XXX; varies on units>148 cfs which are cast in place | 2.7 to 12.3 | 29 to 36 | 0.85 |
| Downstream Defender™ | 0.75 to 13 cfs. | Annual with observations as to whether frequency should be changed. | 19 to 235 $ft^3$ | 70 to 1050 gallons | 18 to 25 | 75 to 93 | 1[6] |
| Stormceptor® | 0.65 to 2.5 cfs. | Annual with observation as to whether frequency should change. Remove when sediment exceeds 15% of sump | 14 to 141 $ft^3$[3,7] | 280 to 1100 gallons | 22 to 66 | 441 to 832 | 0.14 to 0.28[7] |
| StormFilter® | 0.03 cfs to open | Replace cartridges annually | NA as maintenance frequency driven by cartridges rather than sediment accumulation in vault. Generally, the hydraulic action of the cartridge will tend to prevent accumulation of sediment immediately around the cartridge base (Lenhart, pers. comm.) | | | | |
| StormTreat™ | NA | Annual inspection, remove sediment from the StormTreat every 3 to 5 years | Storage volume not defined. There is about 25 $ft^3$ in the bottom of the center well beneath the invert of the inlet that can serve as sediment storage. The unit should be preceded by a manhole with sump to remove coarse sediments, as suggested in manufacturer's specification sheet. | | | | |
| Vortechs™ | 1.6 to 25 cfs | Make quarterly inspections during first year and set frequency accordingly | 20 to 189 $ft^3$ | 270 to 2500 gallons | 7.6 to 12.7 | 100 to 170 | 0.5 to 0.95[6] |
| V2B1™ | 2.8 to 25.2 cfs | Information requested; not yet received. | Information requested | Information requested | | | |

1. Some of the enclosed information provided by personal communication and is not present in the manufacturer's brochure material.
2 Capacity at the point of recommended maintenance. Total capacity is determined by dividing the volume in this column by the % value in the last column .
3. Unit capacities do not necessarily increase with increasing capacity depending on the particular product; most decrease with increasing model size.
4. When the unit is cleaned as a percentage of the total sump volume.
5. Based on data from Tables A2 and 3.4, not D2, in the *Technical and Design Manual*, and letter to RPA of 1/26/99 with recommendation that unit be cleaned when 2' of sediment has accumulated.
6. Storage area is below the vortex unit.
7. Unit capacities based on Table 7 in Technical Manual which gives recommended cleanout based on depth, not %. % based on comparing Table 7 to Table 3.

drainage area. Regardless, the sediment capacity can be included in the consideration of which product to select for a particular site. The relationship between flow capacity and maintenance capacity differs between the products.

E1-2.11.7 Cost

A summary of cost information is presented in Table E1-42. The costs should be considered planning level costs and may differ significantly for a particular site. The costs also do not reflect what would likely be the more difficult and therefore expensive conditions faced with the retrofitting of ultra urban areas or highways. The cost per cfs treated varies considerably between the eight products; however, since the products do not provide exactly the same level of treatment, this can not be converted directly to cost per pound of constituent removed.

E1-2.11.8 References

Greb, S.R., and S. Corsi, 1998, Evaluation of Stormceptor and Multi-Chamber Treatment Train as Urban Retrofit Strategies, Wisconsin Department of Natural Resources, Draft, March 1998 (date of fax, no date on the report).

## E1-2.12 Inlet Filter Inserts

E1-2.12.1 Introduction

Inlet Filter Inserts are designed to be place in existing drain and curb inlets. These devices use various sorbent material and baskets to trap sediment, oil/grease, and litter. One of the primary selling points of the inserts is the low initial cost. There are numerous types of filter inserts including:

- CLR Filter™
- Drainpac™ (http://www.drainpac.com/)
- FossilFilter™ (http://www.kristar.com/faq.html)
- EnviroDrain™ (http://www.enviro-drain.com)
- Gullywasher™ basket (http://www.gullywasher.com/)
- Gullywasher™ “sock” (http://www.gullywasher.com/)
- HydroKleen™ (http://www.stormwater-products.com/hydrokleen.htm)
- Multi-cell™ Filter
- SIFT™ Filter (www.commclean.com)

## Table E1-42 Summary of Cost Information[1] (2/1/99)

| System | Treatment Flow Capacities | Range of Product Cost | Range of Installation Cost or Percentage | Total Construction Cost/CFS |
|---|---|---|---|---|
| BaySaver™ | 2.4, 7.2, and 11.1 cfs. | $7,000 to $10,000 for smallest model; $13,000 to $20,000 for largest | 30% to 50% | $3,800 to $6,250 for smallest unit; $1,500 to $2,700 for the largest unit. |
| CDS™ | 1.1 to 300 cfs, 11 models | $9,600 to $322,500[2] | $13,000 to $630,000[2] | $11,800 for smallest unit; $2,100 for the largest unit. |
| Downstream Defender™ | 0.75, 3, 7, and 13 cfs. | $10,200 to $24,300[3] | 70% to 100% | $23,000 to $33,000 for smallest unit; $3,200 to $4,500 for the largest unit. |
| Stormceptor® | 0.65, 1.1, 1.8 and 2.5 cfs. | $7,600 to $33,560[4] | not provided assumed as 50% to 100% | $17,500 to $23,400 for smallest unit; $8,900 to $11,900 for the largest unit. |
| StormFilter® | 0.03 cfs to open | $8,000± and up | $20% (precast) | $30,000[6] |
| StormTreat™ | 1,400 gallons/unit | $5,600[5] | $750 to $1500 per unit | |
| Vortechs™ | 1.6 to 25 cfs, nine models | $10,500 to $40,000 | 25% to 50% | $8,200 to $9,800 for smallest unit; $2,000 to $2,400 for the largest unit. |
| V2B1™ | 2.8 to 25.2 cfs, seven models | $8,000 to $11,000 for smallest model; $30,000 to $40,000 for largest | 50% | $4,300 to $5,900 for smallest unit; $1,800 to $2,400 for the largest unit. |

1. Excludes costs associated with excessive excavation depths, non-normal construction problems, and other than normal ballast requirements; some of the enclosed information provided by personal communication and is not present in the manufacturer's brochure material.
2. Costs for retrofits; installations in new drainage systems may be less.
3. Standard delivery costs.
4. Excludes delivery costs.
5. Delivery to California.
6. A general average provided by StormWater Management.

- Storm Klenz™
- StreamGuard™ (http://www.parkersystemsinc.com/lc_barriers.htm
- StreamSentry™ (http://www.advenvironmental.com/streamsentry.html)
- Ultra-Urban™ Filter (http://www.abtechindustries.com/UUF.htm)

A brief description of each technology, its manufacturer, and the pollutants that are removed according to the manufacturer, and the installation history are presented in Table E1-43. The total number of installations in the United States and Canada and in the Caltrans system is shown.

The configurations vary significantly between the products and can be grouped as follows:

- *Flow through baskets:* DrainPac, Gullywasher
- *Flow through boxes:* Ultra-urban, HydroKleen
- *Trickle down trays:* EnviroDrain
- *Perimeter tray with open center for high flows:* Fossil Filter, StormKlenz, CLR Filter, SIFT
- *Fabric sock:* Gullywasher, StormGuard, StreamSentry
- *Radial flow cylinder:* Multi-cell

The types of material used to construct the insert body vary. Two products use galvanized metal. Before widespread application of either of these two devices occurs, the question of zinc emissions needs investigation. For most of the products there are no data on how the flow rate through the treatment area decreases over the maintenance

## Table E1-43 Product Descriptions and Installation History

| Insert | Manufacturer | Description | Units Installed | Used by Caltrans |
|---|---|---|---|---|
| CLR Filter | Stormwater Systems, Inc Sacramento, California | Cylinder that is placed in a standpipe connected to the basin outlet. Water enters laterally around the perimeter at the top and flows down through media placed in a concentric ring. High flows pass through the center of the cylinder from the top. Sediments settle in the catch basin. Media is rubberizer. | 4 | 0 |
| DrainPac | United Pumping City of Industry, California | Rectangular, "wire" basket with fabric "bag" takes the form of the basket. The upper end of the basket is bolted to the frame of the catch basin with plates. There is a bypass outlet on each side of the basket at the top. | 125+ | 20+ |
| Envirodrain | Envirodrain, Inc Snohomish, Washington | | | |
| Fossil Filter | Kristar Enterprises Cotati, California | Rectangular, square or circular body with an upper removable tray. Tray is trough extended around the circumference of the catch basin. Open in the center for high flows. Activated alumina media placed in tray. | 5,000 - 6,000 | Being evaluated |
| Gullywasher | AquaTreatment Seattle, Washington | Two basic models: wire mesh basket (round or rect.), and porous fabric "sock" attached to a frame. Sorbent polymer or Absorbent W for o/g. | 2,200 | 0 |
| HydroKleen | Weaver Manufacturing Oroville, California | Two types: box and tapering cylinder. Box: water directed to vertical chamber on one side for sediment. Water overflows to second chamber where it falls through media. Tapering unit collects sediment in perimeter trough; water overflows to center to pass downward through media. | | |
| Multi-cell Filter | Best Management Technologies Crockett, California | Vertical cylinder with 2 concentric rings of media with open center for water outlet and overflow, connected to the basin outlet. Water moves horizontally through the cells. The inner well is open at the top providing for high flow overflow. Sediments settle in catch basin. Several undefined media | 20+ | 1 |
| SIFT Filter | REM Environmental Marketing Antioch, California | Rectangular body with a removal top tray. Body and tray have trough around the perimeter. Top tray for debris; bottom trough for the media. Bypass through center. Activated alumina for media. | 400 - 500 | 0 |
| Storm Klenz | Best Management Technologies Crockett, California | Square or rectangular unit . Has a tray or "gutter" fitted around the circumference into which the water enters, entering a low tray with media. Unit is open in the center for high flows. Undefined media. | undefined | 0 |
| StreamGuard | Foss Environmental Seattle, Washington | Porous fabric container shape of a tapered "sock", sealed at the bottom, attached to a stiffer fabric "frame" trimmed to fit under catch basin grate. Different versions made for sediment, debris, or oil/grease capture. O/g model contains a filter bag of sorbent polymer media. | 20,000+ | unknown |
| StreamSentry | Advanced Environmental Solutions, Seattle, Washington | Essentially the same as the StreamGuard. Sorbent polymer for o/g. | 500 - 1,000 | 0 |
| Ultra-Urban Filter | AbTech Industries Scottsdale, Arizona | Rectangular box for side curb inlet catch basins. Media in bags attached to two sides and bottom through which water passes. Sorbent polymer for media. | 10+ | 0 |

1. As stated by the manufacturer.

cycle; how the accumulation of sediment and debris may reduce the flow rate. This is of particular concern for those inserts that do not possess a specific means for high flow bypass (EnviroDrain) or those where water must pass through a bypass that may be blocked by debris (the flow through boxes and the fabric socks). However, debris blockage may not be an issue where the water first passes through a catch basin grate that retains the larger litter.

A variety of sorbent media is in use. Some of the media are known to rapidly deteriorate; specifically the cellulose media. Some of the media may leach previously retained petroleum products when the concentration drops in the incoming stormwater. However, the information in this regard is incomplete. The media vary in cost, with the cellulose and polypropylene being generally the cheapest. The copolymer rubberizer tends to be the most expensive.

The sorptive processes differ with the media. Polypropylene adsorbs petroleum hydrocarbons; that is, the hydrocarbons remain on the surface of the media. In contrast, the polymers absorb petroleum hydrocarbons; that is, the hydrocarbons “move” into the matrix of the polymer. However the ability of polymers to sorb oil in stormwater may be inhibited.

Treatment flow capacity is the peak rate to which the unit treats stormwater before stormwater enters the bypass. The hydraulic capacity is the maximum flow that can be passed through the unit including the bypass. Some of the manufacturers have not identified the treatment capacity or the hydraulic capacity, or both. Of those that have, few have conducted confirming tests.

### E1-2.12.2 Advantages

- Small footprint and below grade installation for retrofit situations
- Can be highly effective for trash and debris

### E1-2.12.3 Limitations

- Often less effective for removal of conventional stormwater pollutants
- Proprietary products may be difficult for many agencies to specify
- May be locked into a sole source for replacement material
- No reduction of peak flows or runoff volume
- May result in hundreds of individual installations that must be maintained

### E1-2.12.4 Siting and Operational Considerations

These devices are generally designed to be installed in existing drain and curb inlets; consequently, initial cost of this technology is extremely low in comparison to many other alternatives. The potential for implementation of these devices is very high given that sufficient pollutant removal can be demonstrated. Important operational considerations include potential clogging of the device with litter and debris, which can reduce the hydraulic capacity of the inlet and result in street flooding. The hydraulic capacity also limits the maximum drainage area to a given inlet.

A summary of hydraulic testing of the inlet filters is shown in Table E1-44. None of the products have been extensively tested for hydraulic capacity. Only four have been evaluated both before and after placement in the field: EnviroDrain, Gullywasher basket, StreamGuard, and the Ultra-Urban. The Ultra-Urban is the only system for which it has been demonstrated that the design capacity is maintained through the maintenance cycle.

The EnviroDrain unit has been found to quickly clog. The stated hydraulic goal in this study was 167 gpm, the estimated capacity without ponding of a grate in a 20- x by 24-inch catch basin. Controlled flow tests with a fire hydrant determined that the EnviroDrain was capable of passing only 118 gpm without bypass. When placed in the field the media appeared to be rapidly coated with litter and fine sediments which reduced the flow through the media to only a few gpm. The water would then overflow down the sides of the unit (the EnviroDrain does not have a specific bypass route). This problem has been observed in the field.

The Gullywasher basket was similarly affected in tests. Its capacity exceeded 167 gpm initially but was quickly reduced after placement in the field for a short period. The StreamGuard was not as adversely affected. However, its maximum flow rate after field exposure could not be determined because the second set of flow tests was limited to a maximum rate of 24 gpm.

The DrainPac and the Ultra-Urban products have been evaluated for their flow capacities both before and after exposure in the field. Stenstrom evaluated the effect of litter accumulation on head loss. With a clean insert there was no pooling of water at the test flow rate of 200 gpm. At an accumulated rate of 2 kg of litter, the level of the water had “backed up” to the height of the bypass, which was 10”. At a flow rate of 35 gpm, it took about 8 kg of accumulated litter before the water level reached the bypass. The volume used by these quantities of litter was not noted in the reports, preventing any conclusions regarding when the product should be cleaned.

An Ultra-Urban filter was evaluated before and after placement in Santa Monica for seven months. The design flow rate of 35 gpm was achieved both before and after the litter was removed. The unit was about 75% full at the time of these tests.

Another consideration in siting these devices is the potential for a portion of the water to bypass the treatment area of some products even at flows below the available capacity. This can occur by one of three ways.

**Table E1-44 Determination of Hydraulic Capacities**

<table>
<tr><th rowspan="2">Insert</th><th colspan="2">Determination of Hydraulic Capacities</th></tr>
<tr><th>Procedure</th><th>Comments</th></tr>
<tr><td>CLR Filter</td><td>Engineering calculations</td><td>No confirmation with flow tests.</td></tr>
<tr><td>DrainPac</td><td>Lab test</td><td>Tested unit with capacity of 380 gpm but lab test flow rate limited to 200 gpm, which the unit passed without pooling. Capacity dropped with the accumulation of litter.</td></tr>
<tr><td>EnviroDrain</td><td>Lab test</td><td>118 gpm with unit having course screen and one tray with Absorbent W, less than the grate capacity of 167 gpm. Dropped to less than 10 gpm after unit was in the field for a short time.</td></tr>
<tr><td>Fossil Filter</td><td>Field test</td><td>Flow exceeded 12 gpm / lineal foot of insert perimeter, the stated capacity; possibly 20 gpm / lf. Some water observed to travel along grate bars and drop directly into center bypass.<br>Flow was observed to enter primarily on one side. Overflow occurred in the first storm (0.40”) but not in the last three storms (0.04 to 0.12”).</td></tr>
<tr><td>Gullywasher basket</td><td>Lab test</td><td>Exceeded 167 gpm, the grate capacity. Dropped to less than 10 gpm after unit was in the field for a short time.</td></tr>
<tr><td>Gullywasher “sock”</td><td colspan="2">The fabric media has been tested by the manufacturer of the fabric following ASTM procedures.</td></tr>
<tr><td>HydroKleen</td><td colspan="2">No flow tests have been conducted to confirm stated capacities.</td></tr>
<tr><td>Multi-cell Filter</td><td>Field test with water truck</td><td>Tested up to 80 gpm, less than the stated capacity.</td></tr>
<tr><td>SIFT Filter</td><td colspan="2">No flow tests have been conducted to confirm stated capacities.</td></tr>
<tr><td>Storm Klenz</td><td>Field test with water truck</td><td>Tested up to 80 gpm, less than the stated capacity.</td></tr>
</table>

| Insert | Determination of Hydraulic Capacities | |
|---|---|---|
| | Procedure | Comments |
| StreamGuard | | The fabric media has been tested by the manufacturer of the fabric following ASTM procedures. In the ICBIC (1995), exceeded 167 gpm, the grate capacity. Remained above 24 gpm, the maximum available test rate, after placement in the field for a few months. |
| StreamSentry | The fabric media has been tested by the manufacturer of the fabric following ASTM procedures. | |
| Ultra-Urban Filter | Field test of used unit | Unit had been in the field for seven months. Design flow rate of 35 gpm was achieved both before and after the litter was removed. |

1. The stormwater will not likely enter the catch basin evenly around its perimeter but rather concentrated at one or two entry points. This aspect is of most relevance to those inserts with center bypasses: Fossil Filter, SIFT, Storm Klenz, and Multi-cell. The concentrated water may overflow into the center well and/or its momentum may cause a portion of it to enter directly into the center well. Although the CLR Filter has a center well it has a splashguard that prevents the direct entry of stormwater into the bypass.

2. The stormwater may flow across the bars of the grate and drop into the center of the catch basin. This aspect is again of most relevance to those inserts with center bypasses.

3. Thirdly, water may enter between the catch basin rim and the insert. This is most relevant to those inserts that are not attached to the catch basin: Fossil Filter, HydroKleen, Gullywasher basket and sock, SIFT, Ultra-Filter.

The final siting consideration is access for maintenance. Although the manufacturers recommend maintenance at intervals ranging from quarterly to annually, more frequent maintenance may be required at some sites, especially during the wet season. Safety considerations when accessing these sites with personnel and equipment may limit their implementation, particularly on freeways.

### E1-2.12.5 Performance

There are several concerns about the effectiveness of catch basin inserts:

- How effective are sorbent media given the small "detention" time of the water as it passes through the media.
- How removal efficiencies decrease over time as sorbent capacity is consumed.

- Whether the sorbent media become coated with fine sediments and / or litter, interfering with the removal of petroleum products
- Whether the media once it removes petroleum products during periods of relatively high concentrations in turn releases the petroleum when influent concentrations decrease to low levels.
- Whether floatables may clog bypasses.

Table E1-45 provides a summary of the studies that have been conducted to date. The summary indicates that little has been done in the way of evaluation by either the manufacturers of catch basin inserts or their purchasers. None of the studies meet the protocol currently used by Caltrans in the study of public domain BMPs.

With the exceptions of the DrainPac and the Ultra-Urban filters, none of the manufacturers have evaluated the fundamental questions of how performance may be affected by flow rate, influent concentration, and degradation of the sorptive capacity of the media. Only the manufacturers of DrainPac and the Ultra-Urban filters have conducted hydraulic tests to confirm flow capacities.

<u>CLR Filter</u>

As this product has just recently entered the market, it has not been tested for pollutant removal or hydraulic capacity. The manufacturer has estimated the flow capacity of its product using standard weir equations. However, because of the complexities of the flow patterns, the possible effect of the media on flow, and uncertainty of the coefficients of the weir equation, flow tests are necessary to determine the treatment rate.

Given the tendency of oil to sorb to sediments, coating of the media may occur which may reduce the effectiveness of the media. This subject requires testing under conditions that are realistic. This product is constructed with galvanized metal, which raises the issue of zinc being leached into the stormwater.

<u>DrainPac</u>

This device has been tested only in the lab with tap water altered with TSS, oil, or contaminated sediment; however the mixture was not representative of stormwater. Another issue with this product is reliance on the polypropylene fabric bag to remove the petroleum hydrocarbons. Leach tests under conditions realistic of the field need to be conducted. The hydraulic capacity of the unit also has not been tested to confirm flow rates or to evaluate effect of accumulated litter on capacity.

EnviroDrain

Studies indicated good initial removal of hydrocarbons, but performance rapidly diminishes after only about an inch of rainfall. There was essentially no removal of other pollutants tested.

## Table E1-45 Studies of Performance of Catch Basin Inserts

| Insert | No. of Studies | Type | Constituents | References | Results | Comments | Tests Met Caltrans Protocol?[1] |
|---|---|---|---|---|---|---|---|
| CLR Filter | No studies conducted to date. | | | | | | |
| DrainPac | 3 | Lab | TSS, litter, o/g, PAH, lead | Stenstrom (Sept 98, Nov. 98, Feb. 99) | O/G, 49 to 86%; TSS, 96%; PAH, 65%. The liner type, its age, the flow rate, and the influent concentration appears to affect efficiency. 98% removal of litter. | Tap water altered with TSS, oil, or contaminated sediment. Test of lead removal was with contaminated sediment whose size distribution was not representative of stormwater. | Studies were conducted in the lab |
| EnviroDrain | 2 | Field | TSS, o/g, total and dissolved metals | ICBIC (1995), Woodward Clyde (1996) | TPH efficiency dropped quickly after little rainfall in ICBIC study but efficiency did not drop in the WC study. No effect on other pollutants. Essentially no removal of other pollutants. | ICBIC study was with altered stormwater that may have not been representative. | No, ICBIC in lab although with stormwater; only two storms sampled in WC study. |
| Fossil Filter | | | | LWA (1998) | Slight to moderate removals of total lead and copper, dissolved zinc, chlorpyrifos, TSS and diesel TPH. No removal of dissolved lead and copper, total zinc, SVO, motor oil TPH, and diazinon. | | No, only four storms sampled and only three analyzed for hydrocarbons. |
| Gullywasher basket | | Field | TSS, o/g, total and dissolved metals, TP | ICBIC (1995), Woodward Clyde (1996) | TPH efficiency dropped quickly after little rainfall in ICBIC study but efficiency did not drop in the WC study. No effect on other pollutants | | No, ICBIC in lab although with stormwater; only two storms sampled in WC study. |
| Gullywasher "sock" | No studies conducted to date. | | | | | | |
| HydroKleen | No studies conducted to date. Some sorbent tests in the lab. | | | | | | |
| Multi-cell Filter | | Field | | LWA (1999) | | Influent concentrations of TSS and o/g too low to be conclusive | No |
| SIFT Filter | No studies conducted to date. Some sorbent tests in the lab. | | | | | | |
| Storm Klenz | | Lab | | LWA (1999) | | Influent concentrations of TSS and o/g too low to be conclusive | No |
| StreamGuard | | Field | TSS, o/g, total and dissolved metals | ICBIC (1995), Woodward Clyde (1996) | Removed TPH well in both studies. No effect on other pollutants. | Tests were with polypropylene. Product now uses sorbent polymer. | No, ICBIC in lab although with stormwater; only two storms sampled in WC study. |
| StreamSentry | No studies conducted to date. | | | | | | |
| Ultra-Urban Filter | | Field, lab | TSS, litter, o/g | AbTech Industries, Tech Notes 2 and 3; Sta Monica (1998) | Hydraulic efficiency of units not reduced after seven months of operation. o/g 83% to 91%. | Validity of tests with tap water altered with oil is problematic. Filter evaluated after in field for 2 months. | Studies were conducted in the lab |

1. Defined as field research consisting of flow weighted composite samples taken during at least five storms.

Fossil Filter

This product is the most thoroughly evaluated of any of the catch basin inserts. The insert provided slight to moderate removals of total recoverable lead and copper, dissolved zinc, chlorpyrifos, total suspended solids and diesel hydrocarbons. No significant change was seen for dissolved copper, diazinon, dissolved lead, and total recoverable zinc. A significant increase in fecal coliform concentrations was observed; however, the increase is probably more a function of the inherent variability in coliform sample data than an actual increase in concentrations. Several aspects of this product require further study. They include removal performance, and flow tests after being in the field for several weeks or months. The effect of accumulated litter and sediment on both hydraulic and pollutant removal performance are needed.

Gullywasher Basket

Several aspects of this product require further study. They include removal performance, and flow tests after being in the field for several weeks or months. The effect of accumulated litter and sediment on both hydraulic and pollutant removal performance are needed. Will coating of media used in the Gullywasher basket occur as has been found with other media, and will this coating reduce the removal efficiency of petroleum hydrocarbons?

The above point about media is raised because although the Gullywasher was evaluated in earlier tests, a different polymer sorbent is now used. Although the efficacy of this media has been established with fuel spill cleanup, oil is likely adsorbed to the sediments in stormwater which may effect the manner in which it is removed and retained. Tests under conditions representative of stormwater are needed.

Gullywasher “Sock”

Several aspects of this product require study. They include removal performance, and flow tests after being in the field for several weeks or months. The effect of accumulated litter and sediment on both hydraulic and pollutant removal performance are needed. Will coating of media used in the Gullywasher basket occur as was found with other media, and will this coating reduce the removal efficiency of petroleum hydrocarbons? Tests under conditions representative of stormwater are needed.

A particular issue with this product is the role of the polypropylene fabric bag in the removal the petroleum hydrocarbons. Tests of fresh units likely result in petroleum being removed by both the bag and the polymer media. The relative role of each is unknown. If the polypropylene plays a significant role, consideration must be given to how rapidly its capacity is reached, and whether leaching occurs.

HydroKleen

No testing has been conducted of this unit. Both removal performance and flow tests are lacking. As with the other products, the media needs to be evaluated with regard to the question of coating by fine sediments.

Multi-cell Filter

Testing has been limited to date. The concentrations of the TSS in the test runs were too low in the influent to draw conclusions, generally less than 2 mg/L. Similarly the oil concentration was typically below the detection limit. In point of fact, the effluent concentration was consistently higher (1 to 2 mg/L) than the influent. As with the other products, the media needs to be evaluated with regard to the question of coating by the fine sediments.

The unit ($/gpm) cost of this product may be considerably higher than indicated in Table E1-47. This product may require replacement of the existing catch basin (retrofitting) or in new construction using a catch basin that is larger than currently used by Caltrans. In the retrofit situation, closed space entry procedures would be required to install the filter. The center well may be left clear at the top where overflow enters during extreme events. If litter is of concern, a bonnet with a screen can be placed over this entry point to trap and retain the litter in the vault. The bonnet must be periodically washed off.

SIFT Filter

This product has not been tested in the field. It is constructed with galvanized metal, which raises the issue of zinc being leached into the stormwater. However, the manufacturer currently intends to shift to HDPE, or high-density polypropylene. This has the additional benefit of reducing the weight by half.

If floatables are of concern a smaller secondary tray which is perforated can be attached on the inside wall of the overflow area.

Storm Klenz

This product was tested in conjunction with the Multi-cell filter and therefore the same comments apply. The concentrations of the TSS in the test runs were too low in the influent to draw conclusions, generally less than 2 mg/L. Similarly the oil concentration was typically below the detection limit. As with the other products, the media needs to be evaluated with regard to the question of coating by the fine sediments.

StreamGuard

No field tests have been conducted with the current filter material. The older material was effective for reducing TPH; however, there was no effect on other pollutants.

StreamSentry

No tests have been performed with this product. Since the product is similar to the Gullywasher sock and the StreamGuard, results with these products may be considered when evaluating the StreamSentry. Several aspects of this product require further study. They include removal performance, and flow tests after being in the field for several weeks or months. The effect of accumulated litter and sediment on both hydraulic and pollutant removal performance are needed.

Ultra-Urban Filter

The capabilities of this unit with regard to hydraulic performance have been evaluated through an entire maintenance cycle; unique amongst the 12 products reviewed in this report. The laboratory tests suggest that the media retains its removal capability through at least two months of operation, but the testing should be repeated with water that has been altered with both oil and sediment.

E1-2.12.6 Maintenance

Table E1-46 contains a summary of the maintenance recommendations of each manufacturer. The table indicates that the sediment capacity differs between the products, as well as between the models of each product. Manufacturers were generally not forthcoming with either estimated maintenance costs, or with verifiable field experience regarding frequency. Information was limited to their general recommendations on frequency. But information on maintenance costs provided by the vendors may be of little relevance to Caltrans given that Caltrans, unlike most purchasers of these products, does its own maintenance.

With exceptions, inserts have little capacity for sediments. The exceptions are the CLR Filter and the Multi-cell Filter, both of which use the catch basin for sediment storage. Other inserts that have a bit more storage capacity are DrainPac, HydroKleen, and Ultra-Urban.

Most of the products use a media that is white. Therefore its discoloration could be used as an indicator for replacement, although the manufacturers do not make this suggestion in their literature.

## Table E1-46 Summary of Maintenance Information

| Insert | Treatment Capacity (gpm) | Manufacturer's Recommendations | Sediment Storage $ft^3$ | Sediment Storage $ft^3$/gpm | Observations |
|---|---|---|---|---|---|
| CLR Filter | | Periodic inspection, replacement of media, and / or removal of sediment | Capacity of the catch basin | | |
| DrainPac | | Clean once or twice annually depending on site conditions. Clean with vactor truck. | 3 to 36 | | |
| EnviroDrain | | | | | Multi-tray unit when wet and with sediment is heavy. Accumulated litter may hinder flow, resulting in flooding of pavement. |
| Fossil Filter | 100 to 200 | Three cleanings per year; replace media annually. | minimal[1] | | |
| Gullywasher basket | | Inspect three times per year, clean as necessary. | 2 ± | | |
| Gullywasher "sock" | 120 | No recommended procedures | 1 ± | 0.01± | Replacing 4 times per year, for example, costs about $400 |
| HydroKleen | 45 to 405(Mod 200)<br>90 to 200 (Mod 100) | Replace primary sorbent every three to four months; secondary sorbent annually. Remove sediment when chamber 1/3 full. | 1 to 9 (Mod 200)<br>0.3 to 0.5 (Mod 100) | 0.02±<br>0.003± | |
| Multi-cell Filter | 75 to 300 | Experience too limited. | Capacity of the catch basin | | Replacement of media requires removal of insert which may require closed-space entry procedures. |
| SIFT Filter | 50 to 150 | Clean and replace media when 50% coated with contaminants | 0.8± to 2± | 0.015± | |
| Storm Klenz | 80 to 150 | Three filter changes per year | 1.75 | 0.01 to 0.02 | |
| StreamGuard | 120 | Clean debris or sediment models when about one-quarter (6") full. Replace oil unit every 3 to 6 months | 1 ± | 0.01± | Replacing 4 times per year, for example, costs about $400 |
| StreamSentry | 120 | No recommended procedures | 1 ± | 0.01± | Replacing 4 times per year, for example, costs about $400 |
| Ultra-Urban Filter | 35 | Clean sediment and litter quarterly depending on accumulation. Replace sorbent media annually. | 1.5± | 0.04± | |

1. As media is placed in the top tray, the sediment capacity is limited to interstitial space in the media.

### E1-2.12.7 Cost

Cost information for each of the inserts is presented in Table E1-47. The cost data should be considered planning level estimates. Comparing the costs of the products is best done by normalizing the cost to "$ per cfs" of capacity. While a comparing the cost of inserts is important, the utility of the information in Table E1-47 is limited in several respects. The installation costs are not identified for some of the products. The installation costs for the CLR Filter and the Ultra Urban Filter can be expected to be greater than the other filters if the existing catch basin must be replaced with a larger unit. The CLR Filter requires a 3-inch by 3-inch, 4-foot-deep catch basin. The cost for the replacement basin is about $1,200 excluding installation.

**Table E1-47 Capital Cost Information (2/1/99)**

| Insert | Treatment Capacity (Gpm) | Capital Cost | | |
|---|---|---|---|---|
| | | Product | Installation | $ Per Gpm |
| CLR Filter | 300 (18" unit) | $1,050[1] (18" unit) | nominal[1] | $3.50 |
| DrainPac | Units priced by $ft^3$ of sediment storage | $55 to $650 | $115 to $140 | |
| Envirodrain | | | | |
| Fossil Filter | 50 to 190 | $425 to $975 | $25 | $7.50 for smallest unit; $3.80 for largest unit. |
| Gullywasher basket | | | | |
| Gullywasher "sock" | 250 | $90±[2] | Nominal | $0.35 |
| HydroKleen | 45 to 405 | | | |
| Multi-cell Filter | 75 to 300 | $700 to $1235 depending on model and quantity. | Not provided; unlikely to be nominal. | $9.30 for smallest unit; $4.10 for the largest unit, excluding installation. |
| SIFT Filter | 50 to 150 | $350 to $650 | Likely nominal | $7.00 for smallest unit; $4.30 for the largest unit |
| Storm Klenz | 80 to 150 | $305 to $700 depending on model and quantity. | Likely nominal | $3.80 for smallest unit; $4.70 for the largest unit. |
| StreamGuard | 250 | $90±[2] | Nominal | $0.35 |
| StreamSentry | 250 | $90±[2] | Nominal | $0.35 |
| Ultra-Urban Filter | 35 per module | $250 per module | Not provided; unlikely to be nominal | $6.70 per module excluding installation |

1. Does not include installation of standpipe in existing catch basin or replacement of a basin that is too small. See text. Includes cost of media which is purchased separately. The empty filter is $900 (Stetler, pers. comm.)
2. Oil sorption model; sediment models are less.

Several of the products including the Gullywasher "sock," StormGuard, and StormSentry are "throw-aways," particularly if they are used to remove petroleum hydrocarbons. They are supposed to be frequently replaced, on the order of several times per year. These products can be reused if they are only used to remove sediment; at a construction site for example. Finally, the performance likely varies between the products. Consequently, a product may be more expensive but it may have better performance.

#### E1-2.12.8 References

Interagency Catch Basin Insert Committee (ICBIC), October 1995. *Evaluation of Commercially Available Catch Basin Inserts for the Treatment of Stormwater Runoff from Developed Sites.* (ICBIC consisted of City Seattle, King County, Snohomish County, Port of Seattle, Washington).

Larry Walker Associates, 1998. "NDMP Inlet / In-Line Control Measure Study Report." Report to the City of Sacramento.

Larry Walker Associates, 1999. "Guidance Document on BMP Evaluation and Selection," Caltrans Contract No. 43A005, Task 5 Review of Catch Basin Inserts.

Santa Monica (City of), 1998. "Santa Monica Bay Area Municipal Storm Water / Urban Runoff Pilot Project - Evaluation of Potential Catchbasin Retrofits," prepared by Woodward Clyde.

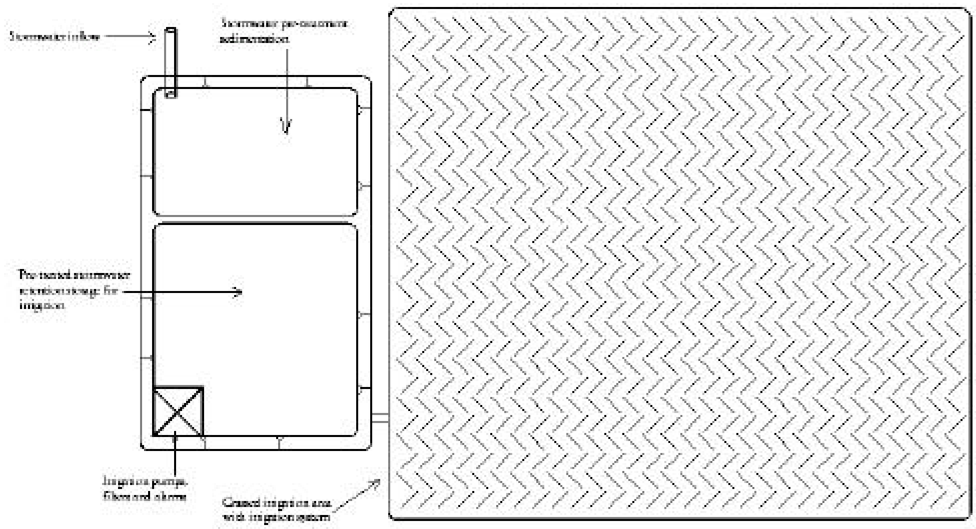
Stenstrom, M.K. Three letter reports to Mr. Paul Corn of United Pumping Services, dated September 25, 1998, November 17, 1998, and February 5, 1999.

Woodward Clyde, 1996 "Parking Lot Monitoring Report," Santa Clara Valley Nonpoint Source Pollution Control Program.

### E1-2.13 Retention/Irrigation

#### E1-2.13.1 Description

Retention / irrigation refers to the capture of stormwater runoff in a holding pond and subsequent use of the captured volume for irrigation of landscape or natural pervious areas. This technology is very effective as a stormwater quality practice in that, for the captured water quality volume, it provides virtually no discharge to receiving waters and high stormwater constituent removal efficiencies. This technology mimics natural undeveloped watershed conditions wherein the vast majority of the rainfall volume during smaller rainfall events is infiltrated through the soil profile. Their main advantage over other infiltration technologies is the use of an irrigation system to spread the runoff over a larger area for infiltration. This allows them to be used in areas with low permeability soils.




Capture of stormwater can be accomplished in almost any kind of runoff storage facility, ranging from dry, concrete-lined ponds to those with vegetated basins and permanent pools. The pump and wet well should be automated with a rainfall sensor to provide irrigation only during periods when required infiltration rates can be realized. Generally, a spray irrigation system is required to provide an adequate flow rate for distributing the water quality volume (LCRA, 1998). Collection of roof runoff for subsequent use (rainwater harvesting) also qualifies as a retention/irrigation practice.

This technology is still in its infancy and there are no published reports on its effectiveness, cost, or operational requirements. The guidelines presented below should be considered tentative until additional data are available.

### E1-2.13.2 Advantages

- Pollutant removal effectiveness is high, accomplished primarily by: (1) sedimentation in the primary storage facility; (2) physical filtration of particulates through the soil profile; (3) dissolved constituents uptake in the vegetative root zone by the soil-resident microbial community.

- The hydrologic characteristics of this technique are effective for simulating pre-developed watershed conditions through: (1) containment of higher frequency flood volumes (less than about a 2-year event); and (2) reduction of flow rates and velocities for erosive flow events.
- Pollutant removal rates are estimated to be nearly 100% for all pollutants in the captured and irrigated stormwater volume. However, relatively frequent inspection and maintenance is necessary to assure proper operation of these facilities.
- This technology is particularly appropriate for areas with infrequent rainfall because the system is not required to operate often and the ability to provide stormwater for irrigation can reduce demand on surface and groundwater supplies.

### E1-2.13.3 Limitations

- Retention-irrigation is a relatively expensive technology due primarily to mechanical systems, power requirements, and high maintenance needs.
- Due to the relative complexity of irrigation systems, they must be inspected and maintained at regular intervals to ensure reliable system function.
- Retention-irrigation systems use pumps requiring electrical energy inputs (which cost money, create pollution, and can be interrupted). Mechanical systems are also more complex, requiring skilled maintenance, and they are more vulnerable to vandalism than simpler, passive systems.
- Retention-irrigation systems require open space for irrigation and thus may be difficult to retrofit in urban areas.
- Effective use of retention irrigation requires some form of pre-treatment of runoff flows (i.e., sediment forebay or vegetated filter) to remove coarse sediment and to protect the long-term operating capacity of the irrigation equipment.

### E1-2.13.4 Siting

This technology is implemented where no discharge of stormwater is the objective, but where Hydrologic Class C and D soils prevent their implementation.

### E1-2.13.5 Design

- *Runoff Storage Facility Configuration and Sizing* - Design of the runoff storage facility is flexible as long as the water quality volume and an appropriate pump and wet well system can be accommodated.

- *Pump and Wet Well System* - A reliable pump, wet well, and rainfall or soil moisture sensor system should be used to distribute the water quality volume. These systems should be similar to those used for wastewater effluent irrigation, which are commonly used in areas where "no discharge" wastewater treatment plant permits are issued.

- *Detention Time* - The irrigation schedule should allow for complete drawdown of the water quality volume within 72 hours. Irrigation should not begin within 12 hours of the end of rainfall so that direct storm runoff has ceased and soils are not saturated. Consequently, the length of the active irrigation period is 60 hours. The irrigation should include a cycling factor of ½, so that each portion of the area will be irrigated for only 30 hours during the total of 60 hours allowed for disposal of the water quality volume. Irrigation also should not occur during subsequent rainfall events.

- *Irrigation System* - Generally a spray irrigation system is required to provide an adequate flow rate for timely distribution of the water quality volume.

- *Irrigation Site Criteria* – The area selected for irrigation must be pervious, on slopes of less than 10%. A geological assessment is required for proposed irrigation areas to assure that there is a minimum of 12 inches of soil cover. Rocky soils are acceptable for irrigation; however, the coarse material (diameter greater than 0.5 inches) should not account for more than 30% of the soil volume. Optimum sites for irrigation include recreational and greenbelt areas as well as landscaping in commercial developments. The stormwater irrigation area should be distinct and different from any areas used for wastewater effluent irrigation. Finally, the area designated for irrigation should have at least a 100-foot buffer from wells, septic systems, and natural wetlands.

- *Irrigation Area* – The irrigation rate must be low enough so that the irrigation does not produce any surface runoff; consequently, the irrigation rate may not exceed the permeability of the soil. The minimum required irrigation area should be calculated using the following formula:

$$A = \frac{12 \times V}{T \times r}$$

where:
$A$ = area required for irrigation (ft$^2$)
$V$ = water quality volume (ft$^3$)
$T$ = period of active irrigation (30 hr)
$r$ = Permeability (in / hr)

- The permeability of the soils in the area proposed for irrigation should be determined using a double ring infiltrometer (ASTM D 3385-94) or from county soil surveys prepared by the Natural Resource Conservation Service. If a range of permeabilities is reported, the average value should be used in the calculation. If no permeability data is available, a value of 0.1 inches/hour should be assumed.

- It should be noted that the minimum area requires intermittent irrigation over a period of 60 hours at low rates to use the entire water quality volume. This intensive irrigation may be harmful to vegetation that is not adapted to long periods of wet conditions. In practice, a much larger irrigation area will provide better use of the retained water and promote a healthy landscape.

### E1-2.13.6 Performance

This technology eliminates the routine discharge of stormwater and 100% of the associated pollutants to surface waters.

### E1-2.13.7 Maintenance

- The irrigation system should be inspected and tested (or observed while in operation) to assure proper operation at least 6 times annually. Two of these inspections should occur during or immediately following wet weather. Any leaks, broken spray heads, or other malfunctions with the irrigation system should be repaired immediately.

- Remove sediment from inlet structure/sediment forebay, and from around the sump area at least 2 times annually or when depth reaches 3 inches. When sediment in other areas of the basin fills the 10% volume allocated for sediment accumulation, all sediment should be removed and disposed of properly.

- The upper stage, side slopes, and embankment of a retention basin must be mowed regularly to discourage woody growth and control weeds. Grass areas in and around basins must be mowed at least twice annually to limit vegetation height to 18 inches. More frequent mowing to maintain aesthetic appeal may be necessary in landscaped areas. When mowing is performed, a mulching mower should be used, or grass clippings should be caught and removed.

- Debris and litter will accumulate near the basin pump and should be removed during regular mowing operations and inspections. Particular attention should be paid to floating debris that can eventually clog the irrigation system.

- The pond side slopes and embankment may periodically suffer from slumping and erosion, although this should not occur often if the soils are properly compacted during construction. Regrading and revegetation may be required to correct the problems.

### E1-2.13.8 Cost

O&M costs for retention-irrigation systems are high compared to virtually all other stormwater quality control practices because of the need for: (1) frequent inspections; (2) the reliance on mechanical equipment; and (3) power costs.

### E1-2.13.9 References

Barrett, M., 1999. *Complying with the Edwards Aquifer Rules: Technical Guidance on Best Management Practices*, Texas Natural Resource Conservation Commission Report RG-348. http://www.tnrcc.state.tx.us/admin/topdoc/rg/348/index.html

Lower Colorado River Authority, 1998. *Nonpoint Source Pollution Control Technical Manual*, Austin, Texas.

## E1-3 Local Experience

BMPs have been constructed in several areas of Orange County primarily to address the problems related to nutrient loads and bacteria concentrations. These sites include several demonstration projects along Aliso Creek, which was the subject of a Cleanup and Abatement order because of high concentrations of indicator organisms. A number of BMPs have also been constructed in the watershed of Newport Bay, a 303(d) listed waterbody. These BMPs consist primarily of sediment traps located in San Diego Creek and Upper Newport Bay. In addition, the San Joaquin Marsh was constructed near the head of the bay to reduce nutrient loadings. Finally, a series of "Natural Treatment Systems" are proposed for the watershed to further address high bacteria concentrations. Each of these BMPs will be discussed in detail in the following sections.

These local BMPs differ significantly in design and purpose from those described in the previous chapter. The most significant difference is that in the cases where nutrient and bacteria reduction is the objective the facilities are designed primarily to treat dry weather flows. Consequently, these devices can be much smaller than those that would be required under the current permit for treating stormwater flows from the same watershed. Since dry weather flow is present at all of these locations, most of the BMPs are wet ponds or constructed wetlands. It is unlikely that these types of BMPs would find widespread application for onsite treatment because of the lack of perennial flow from most developed watersheds. Because of the differences in treatment objectives and design, the data gained from the BMPs implemented to date in Orange County generally will not help to verify the BMP data from published studies in other areas.

### E1-3.1 Aliso Creek Demonstration Projects

Aliso Creek has been the site of numerous stormwater treatment projects because of the high levels of some constituents of concern, even during dry weather. One important objective is to reduce the concentration of pathogens and other indicator organisms in the discharge to the Pacific Ocean in response to a Cleanup and Abatement Order for the JO3P02 outfall to Sulphur Creek, a tributary of Aliso Creek, located in Laguna Niguel.

#### E1-3.1.1 Wetlands

Two wetlands have been constructed, termed the East and West Wetlands, with a third, the North Wetland, currently under construction (Figure E1-s 34, 35, and 36). The system includes a low-flow inlet and piping system, parallel to the existing storm drains, which will divert virtually all gutter low flows from a 1-square-mile residential area into the three constructed natural treatment wetlands before releasing flows into Sulphur Creek. Both the East and West Wetlands were constructed by enhancing existing wetlands, which in most cases would not be allowed. The enhancements consist principally of installing small berms across the drainage ways to increase residence time and create small pools. In general, there is not sufficient baseflow to support a wet pond and often there is little or no discharge from the wetlands. During these periods essentially 100% removal of bacteria and other constituents is achieved. The existing West Wetland was originally constructed 10 years ago as wetland habitat mitigation for the adjacent residential tracts. The half-mile-long, 1.4-acre West Wetland runs parallel to Alicia Parkway along its west side. The wetland currently receives low-flow runoff water from the southerly 41% of the JO3P02 watershed. The low- flow water is diverted directly out of the main JO3P02 pipe into a smaller pipe which leads to the upper end of the West Wetland. The West Wetland receives water from 63% of the JO3P02 watershed. The West Wetland is located on privately owned common-area property.

**Figure E1-34 Berm in the West Wetland, Laguna Niguel**

The existing East Wetland subsystem was originally constructed in 2001 as a habitat mitigation project for a park project elsewhere in the City. The 0.3-acre wetland, located on the east side of Alicia Parkway about 1/3 mile upstream of JO3P02's Sulphur Creek outfall, currently receives diverted low-flow water from gutters in the east-central area that represents about 14% of the JO3P02 watershed. The East Wetland is located on privately-owned common-area property.

The North Wetland is being developed as a completely new subsystem to treat water from gutters and v-ditches in the northeast part of the JO3P02 watershed, comprising 23% of the total drainage area. North Wetland improvements will include new drain inlets, piping, and tiered wetland pond construction. The North Wetland is located at the northern tip of the JO3P02 drainage area on the east side of Alicia Parkway, on public property within Laguna Niguel Regional Park, which is a regional park owned and operated by the County of Orange.

**Figure E1-35 East Wetland in Laguna Niguel**

Although these wetlands result in small pools of standing water with associated wetland plants, mosquito control has not been an issue to date. This is in stark contract to the experience of Caltrans, where the local vector control agencies in Los Angeles County were vehemently opposed to creation of permanent pools with the drainage infrastructure and required repeated abatement when mosquito breeding was observed.

There has also been no problem associated with threatened or endangered species. None have been observed at any of the wetland sites and no special precautions are taken prior to performing maintenance.

Surprisingly, wetland protection has not been an issue at these sites, despite the fact that these treatment systems were originally dedicated wetland mitigation areas. Normally, stormwater treatment would not be allowed and any activity in these areas would require additional mitigation offsite as a result of conservation easements over the wetland areas. It is highly unlikely that other agencies or developers construction BMPs would be allowed to use mitigation areas for this purpose.

**Figure E1-36 North Wetland under Construction**

Limited monitoring to date indicates dramatic reductions in fecal coliform counts. Data collected during 2001 show have the following average concentrations:

East Wetland Influent (n = 43) 13,965 MPN / 100 ml
East Wetland Effluent (n = 43) 203 MPN / 100ml
West Wetland Influent (n = 11) 5,419 MPN / 100mL
West Wetland Effluent (n = 10) 288 MPN / 100mL

Substantial reductions in total coliforms and Enterococcus were also observed; however average effluent concentrations were still quite high. Similarly, reductions in nitrogen and phosphorus concentrations are evident, although effluent concentrations still exceed the Basin Plan objectives.

### E1-3.1.2 Ultraviolet Disinfection

Another project funded by the City of Laguna Niguel is the Clear Creek Ultraviolet Disinfection Facility. This facility, pictured in Figure E1-37, is a highly mechanical system for filtering and disinfecting dry weather flows at outfall JO3P02. Although the system is highly effective for removing bacteria, as demonstrated by the data in Table E1-48, the cost of this treatment is substantial. The process must constantly be refined to maintain this level of performance and significant electrical power is required for operation. Influent is treated by sand filtration and then exposed to ultraviolet light for disinfection, prior to being discharged to the creek.

**Figure E1-37 Clear Creek UV System**

This is a closed system; consequently there are no vector or endangered species issues. The small footprint of this system facilitates the avoidance of wetland areas, so this has not been an obstacle to its use.

## Table E1-48 Performance of the Clear Creek UV System

| Date | Influent/ Effluent | Total Coliform | Fecal Coliform | E. Coli | Enterococcus |
|---|---|---|---|---|---|
| 3/27/01 | Effluent | 3 | <1 | <1 | <1 |
| 3/27/01 | Effluent | <1 | <1 | <1 | <1 |
| 3/28/01 | Effluent | 3 | <1 | <1 | <1 |
| 3/28/01 | Effluent | <1 | <1 | <1 | <1 |
| 3/29/01 | Effluent | 8 | <1 | <1 | <1 |
| 3/29/01 | Effluent | <1 | <1 | <1 | <1 |
| 4/16/01 | Effluent | 30 | <1 | <1 | <1 |
| 4/17/01 | Effluent | <1 | <1 | <1 | <1 |
| 4/24/01 | Effluent | <2 | <2 | <2 | <2 |
| 5/1/01 | Effluent | <2 | <2 | <2 | <2 |
| 5/8/01 | Effluent | 500 | 300 | 300 | <2 |
| 5/11/01 | **Influent** | 26,000 | 11,000 | not tested | 9,000 |
| 5/11/01 | Effluent | 900 | 170 | not tested | 80 |
| 5/14/01 | Effluent | <1 | <1 | <1 | <1 |
| 5/15/01 | Effluent | <2 | <2 | <2 | <2 |
| 5/16/01 | Effluent | 10 | <1 | <1 | <1 |
| 5/17/01 | **Influent** | 40,000 | 5,000 | 5,000 | 4,000 |
| 5/17/01 | Effluent | <1 | <1 | <1 | <1 |
| 5/17/01 | Effluent | 6 | <1 | <1 | <1 |
| 5/19/01 | Effluent | 5 | <1 | <1 | <1 |
| 5/21/01 | **Influent** | 120,000 | 9,000 | 8,000 | 8,000 |
| 5/21/01 | Effluent | 30 | <1 | <1 | <1 |
| 5/21/01 | **Influent** | 130,000 | 9,000 | 8,000 | 9,000 |
| 5/21/01 | Effluent | 150 | 9 | <1 | 9 |
| 6/5/01 | Effluent | 70 | <2 | <2 | <2 |
| 6/5/01 | Effluent | 90 | 11 | 11 | 21 |
| 6/5/01 | Effluent | 240 | 17 | 11 | 11 |
| 6/15/01 | Effluent | <1 | <1 | not tested | not tested |
| 6/19/01 | Effluent | 17 | <2 | <2 | <2 |
| 6/28/01 | Effluent | 20 | <2 | not tested | not tested |
| 7/03/01 | Effluent | <2 | <2 | <2 | <2 |
| 7/10/01 | **Influent** | 110,000 | 21,000 | 13,000 | 21,000 |
| 7/10/01 | Effluent | 1,600 | 240 | 240 | 1,600 |
| 7/17/01 | **Influent** | 260,000 | 11,000 | 9,000 | 13,000 |
| 7/17/01 | Effluent | 1,600 | 50 | 30 | 500 |
| 7/24/01 | Effluent | <2 | <2 | <2 | <2 |
| 7/31/01 | Effluent | <2 | <2 | <2 | <2 |
| 8/07/01 | Effluent | <2 | <2 | <2 | <2 |
| 8/14/01 | Effluent | <2 | <2 | <2 | <2 |
| 8/28/01 | Effluent | 9 | <2 | <2 | <2 |
| 9/04/01 | Effluent | <2 | <2 | <2 | <2 |
| 9/11/01 | Effluent | <2 | <2 | <2 | <2 |
| 9/18/01 | Effluent | 11 | <2 | <2 | <2 |
| 9/25/01 | Effluent | 50 | <2 | <2 | <2 |

## E1-3.2 Newport Bay Projects

BMPs have been constructed at several locations in the Newport Bay / San Diego Creek watershed, principally in and adjacent to San Diego Creek. These BMPs have been constructed to reduce the discharge of sediment and bacteria to the Bay in response to the 303(d) listing for this waterbody. The objective of the sediment TMDL for the Bay is to reduce the annual average sediment load in the San Diego Creek watershed from a total of 250,000 tons per year to 125,000 tons per year, thereby reducing the sediment load to Newport Bay to 62,500 tons per year within 10 years (a 50% reduction).

### E1-3.2.1 Sediment Trapping Basins

To implement the sediment TMDL, a Cooperative Agreement entitled "Newport Bay / San Diego Creek Watershed Sediment Control Monitoring and In-Channel Maintenance Program" was executed on April 20, 1999. The Agreement between the County of Orange, The Irvine Company and the cities of Newport Beach, Irvine, Tustin and Lake Forest, provides a funding base for the implementing the various components of the sediment TMDL and in-channel basin maintenance programs.

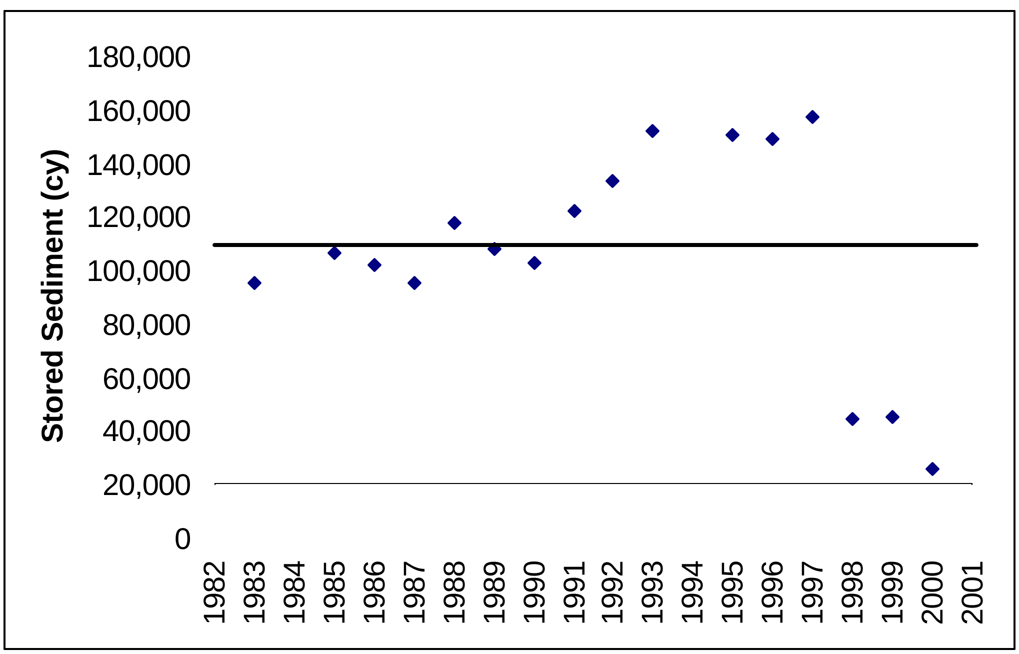
As part of the early action 208 Program, seven foothill water retarding / sediment trapping basins (Hicks Canyon, East Hicks Canyon, Round Canyon, Agua Chinon, Bee Canyon, Marshburn and Orchard Estates) have been completed in the upper San Diego Creek watershed and three in-channel sediment trapping basins in the lower reach of San Diego Creek just upstream of the Bay. The in-channel basins were constructed by placing a low rock dam across the creek as illustrated in Figure E1-38. The capacities of the basins are 218,000 (Basin 1), 64,000 (Basin 2), and 90,800 cubic yards (Basin 3).

To comply with the sediment TMDL, an annual report is submitted to the RWQCB by November 15 of each year verifying that the basins have at least 50% capacity. The 2000-2001 sediment scour study report showed that all basins were in compliance; however, Figure E1-39 indicates that in many prior years in-channel Basin 1 often had less than 50% of its capacity (144,000 $yd^3$) available. Historical records indicate that sediment removal from this basin occurred in 84-85 (12,000 $yd^3$), 93-94 (68,000 $yd^3$), and 99-2000 (35,000 $yd^3$), but little or no increase in available storage was observed, indicating that the annual accumulation could quickly replace the removed sediment. A large increase in available volume occurred between 1997 and 1998 when there was no sediment removal, suggesting that much of the stored sediment was flushed from the basin during larger storm events. These data suggest that annual sediment removal would greatly increase the efficiency of this basin.

No issues related to vectors, threatened or endangered species, or wetland protection have been reported for these basins.

**Figure E1-38 Sediment Basin in San Diego Creek**


**Figure E1-39 Accumulated Sediment in Basin 1**



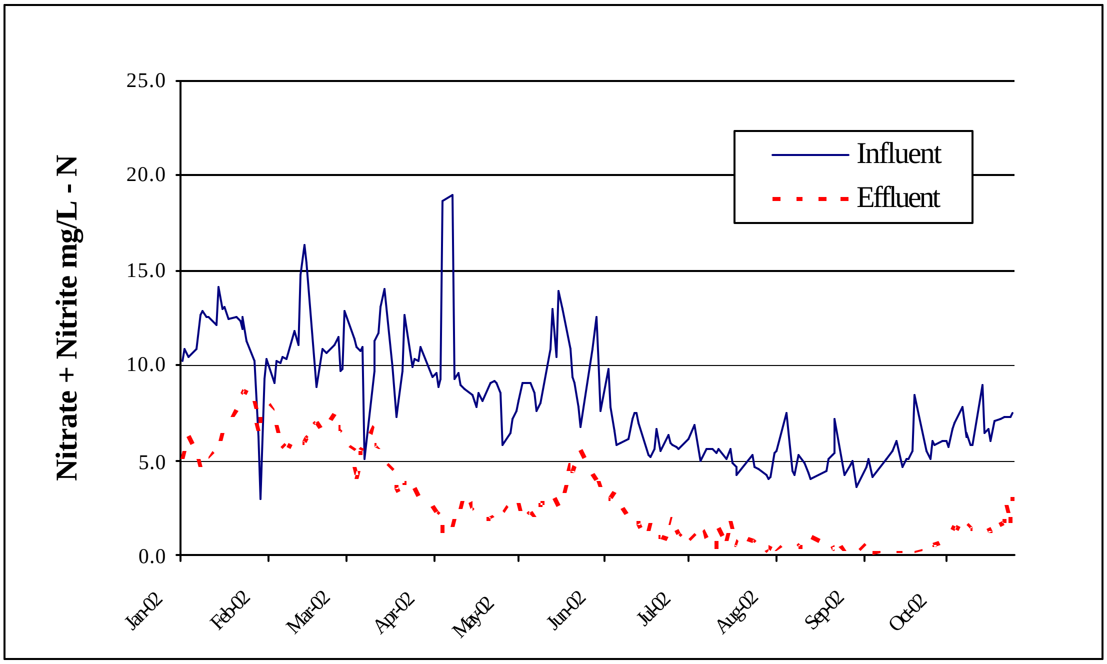
E1-3.2.2 San Joaquin Marsh

The San Joaquin Marsh was constructed by the Irvine Ranch Water District primarily to reduce nutrient concentrations in the San Diego Creek discharge to Newport Bay. A photograph of this facility is shown in Figure E1-40. Dry weather flows in the creek are pumped through a series of ponds before most of the flow is returned to San Diego Creek. A portion of the treated water is used onsite for mitigation projects. A comparison of influent and effluent concentrations for the marsh is presented in Figure E1-41 and shows roughly a 50% reduction in concentration. The annual load reduction of total nitrogen is estimated to be 60,000 to 70,000 lbs/yr. The data also indicate a reduction in average fecal coliform counts from about 4600 CFU/100mL to less than 100 CFU/100mL, which meets the standard for primary contact recreation.

No issues related to vectors or threatened or endangered species has been reported for this site. Wetland mitigation was an issue and a portion of the tract is used to provide that mitigation.

**Figure E1-40 San Joaquin Marsh**

**Figure E1-41 Comparison of Influent and Effluent $NO_3$ Concentrations in 2002**




### E1-3.3 Natural Treatment Systems

One of the most ambitious examples of this strategy will soon be provided by the Irvine Ranch Water District (IRWD) and other stakeholders in the Newport Bay watershed. IRWD provides domestic water service, sewage collection, and water reclamation for the city of Irvine and the unincorporated areas of south-central Orange County as well as portions of Tustin, Santa Ana, Newport Beach, Costa Mesa, Orange, and Lake Forest. The district's service areas drain into Newport Bay primarily through San Diego Creek, both of which the State Water Quality Control Board (SWQCB) has identified as impaired because of contamination by heavy metals, pesticides, and other toxins. Newport Bay also is contaminated by nutrients such as nitrogen and phosphorus, which are not themselves toxic but which cause algae blooms and can lead to fish die-offs as decaying algae deplete the water of oxygen.

The Irvine Ranch Water District (IRWD) plans to construct about 37 small wetlands scattered throughout the San Diego Creek watershed. Dry-weather runoff from existing and new development will be shunted through the network of ponds and marshes, where plants and microbes will absorb nitrogen and other nutrients and break down bacteria and other contaminants. Once it has been cleaned by biological processes, the runoff will be allowed to flow into the existing waterways.

IRWD is calling its project the Natural Treatment System and estimates it will cost $25 million to $30 million to build. It is modeled after an earlier marsh restoration project the district credits with a 25 percent drop in algae blooms in Newport Bay. Beginning in 1996, the district diverted the flow from San Diego Creek into a restored wetlands complex known as San Joaquin Marsh. After circulating through the marsh and its ponds for several days, during which it is filtered by algae, cattails, bulrushes, and other aquatic vegetation, the water returns to the creek channel with half its nitrogen content removed.

IRWD's project, which it is developing in partnership with Orange County and several local cities, is still in the early planning stages. Environmental review is expected to end this month (January 2003) and design is expected to commence after that. If all goes as planned, construction will begin next year (2004). Some of the wetlands will be installed in existing stormwater and flood detention basins. In new development areas, however, the district expects landowners to provide property or easements and to pay for the costs of constructing the wetlands and related facilities. The district also is seeking state and federal grant money to cover part of the cost.

Because the project is still in the early planning stages, no issues related to vectors, threatened or endangered species, or wetland mitigation have been identified.

### E1-3.4 Other/Proposed Projects

#### E1-3.4.1 Dairy Fork Demonstration Projects

Orange County is currently working on the grant scope of work for a vegetated treatment system on Dairy Fork in the Aliso Creek watershed. The County is proposing to build a series of three flow-through stop log and rock biofilters with corresponding upstream basins to provide some flow attenuation and retain suspended solids. Basin margins will be vegetated with emergent marsh vegetation (sedges, rushes, cattails).

## E1-4 Performance Summary

### E1-4.1 Comparison of Performance by Constituent

The individual chapters, which reviewed the performance of the various BMPs, presented all the available performance data irregardless design features, climatic conditions, watershed characteristics, or other variables that might affect reported removal efficiency. In this chapter and in subsequent discussions the data relative to BMP performance will be limited to a subset of the data that is particularly relevant for estimating the pollutant removal that would be expected in Orange County in BMPs designed and sized according to local requirements.

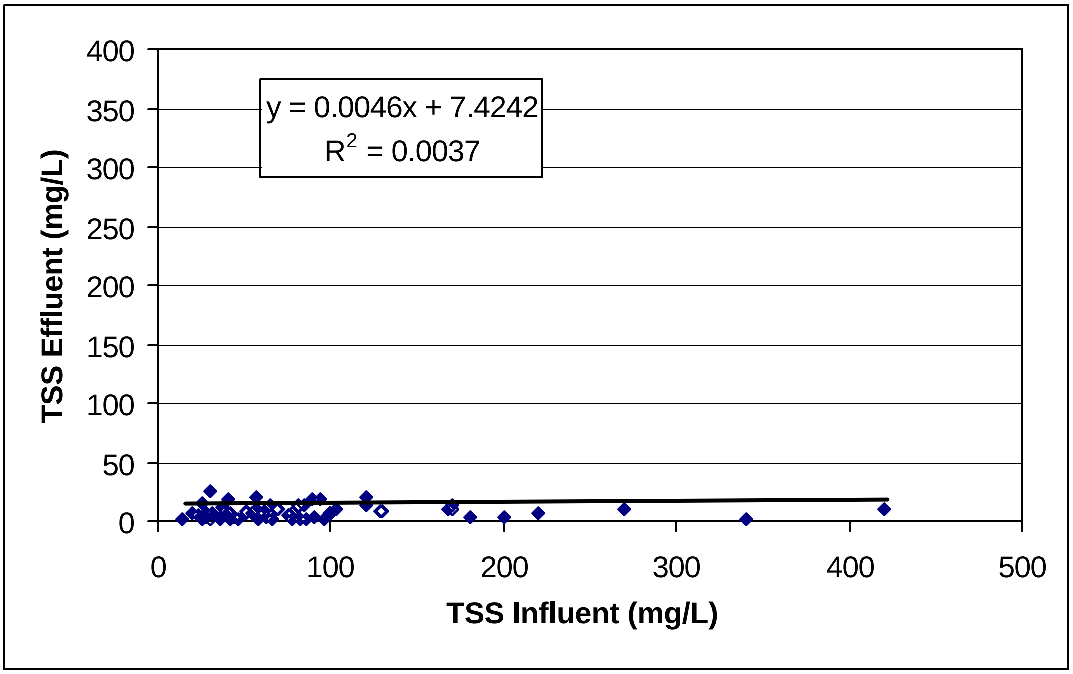
These performance numbers generally reflect the experience gained in the Caltrans BMP Retrofit Pilot Program, which was conducted in the San Diego and Los Angeles areas. A total of 39 BMPs, consisting of 13 different types of facilities were constructed and monitored over a three-year period. As many as 6 individual installations of a single BMP type were implemented, resulting in the monitoring of as many as 65 storms for the facilities with the most sites.

The previous sections often reported pollutant removal performance in the conventional way, as a percent reduction in concentration observed between the BMP influent and effluent. According to Striker et al. (2001), there are at least four common techniques for estimating the pollutant reduction of BMPs. Historically, however, pollutant removal efficiency has been expressed almost universally as a percent reduction in the concentration or load for the constituents of concern based on flow-weighted samples collected of the untreated and treated runoff.

This traditional measure of the performance of BMPs may not truly reflect the relative performance of the devices when the influent concentration is relatively low or when the device functions such that the concentration of the BMP effluent is unrelated to influent concentration. For instance, lower percent removal is often reported for low influent concentrations in BMP monitoring studies (i.e., Horner and Horner, 1999). In addition, the use of percent removal may lead the reader into improperly applying these published “efficiencies” to watersheds with substantially different runoff characteristics where the values are not valid. Another use of “removal efficiency” would be to estimate the discharge quality from an event with particularly high pollutant concentrations to estimate compliance with water quality standards. This chapter will demonstrate that in many cases both of these applications of a published percent reduction in pollutant concentration produce results that do not accurately characterize the expected quality of the treated runoff.

A linear regression analysis was performed on the paired influent and effluent event mean concentrations (EMCs) from each type of device in the Caltrans study to assess whether the pollutant removal performance can be expressed as a percent reduction. Examining the regression analysis results for Austin sand filters can illuminate several striking aspects of the analysis. Figure E1-42 presents the influent and effluent EMCs for TSS. The regression line shown on the graph is not statistically significant at the 90% confidence level, so the effluent EMC is independent of the influent concentration. Therefore the expected TSS concentration of treated runoff is better characterized as a constant value, 7.8 mg/L, with uncertainty at the 90% confidence level of only 1.2 mg/L. This small uncertainty highlights the very consistent performance of sand filters for TSS removal.

**Figure E1-42 Influent and Effluent Concentrations of TSS in Austin Sand Filters**

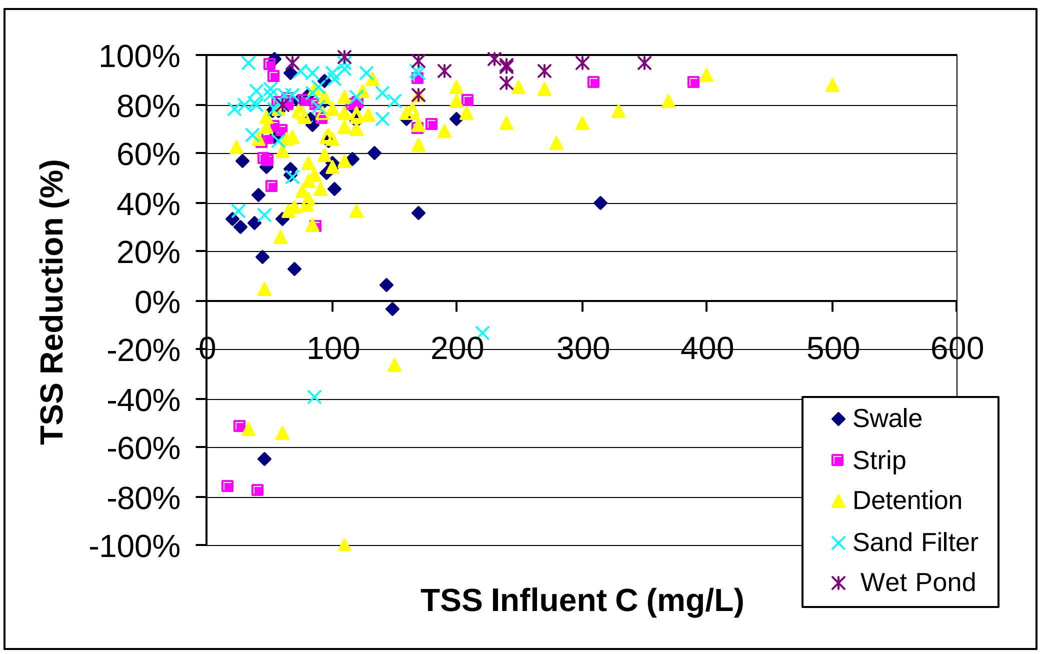



In addition, the constant effluent concentration means that a calculated percent reduction for TSS and other constituents with similar behavior for Austin sand filters is a secondary characteristic of the device and depends primarily on the specific average influent concentrations observed. For instance, a conventional analysis of the sand filter data indicates a removal of about 90% for TSS. This is due primarily to the fact that the average influent concentration was about 75 mg/L. The distinction between a constant effluent quality and a percent reduction is extremely important to recognize if the results are to be used to estimate effluent quality from sand filters installed at other sites with different influent concentrations or for estimating compliance with water quality standards for storms with high concentrations of particulate constituents.

The stable effluent concentration of a sand filter under very different influent TSS concentrations implies something about the properties of the influent particle size distribution. If one assumes that only the smallest size fraction can pass through the filter, then the similarity in effluent concentrations suggests that there is little difference in the total mass of the smallest sized particles even when the total TSS concentration varies greatly. Further, the difference in TSS concentration must then be caused by changes in the relative amount of the larger size fractions. This dependence of removal level on influent particle size suggests that one could expect differences in removal based on differences in particle size distribution in runoff at other sites. However, the TSS reduction observed for sites in the Orange County area is very similar to that observed in previous studies (Glick et al., 1998), which suggests that the differences are not large.

A plot of removal efficiency versus TSS influent concentration of data collected by Caltrans (Figure E1-43) also highlights the variability of calculated reduction in response to changes in influent concentration. One can see that at low concentrations occasional increases in concentration (negative removal) are observed.

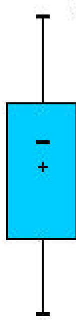
**Figure E1-43 TSS Reduction as a Function of Influent Concentration**




Because the removal efficiency is not constant for a BMP, this chapter compares BMPs based on the quality of the discharge achieved. Box plots are presented for selected constituents to display graphically the effluent quality one might expect. In the plots, the horizontal line in each box represents the median effluent concentration observed. The boxes represent the data range of the 25% of the values that lie above and below the median (i.e., the 2nd and 3rd quartiles). The whiskers extend to the highest value observed that is within 1.5 times the height of the box. Points outside of this range (outliers) are plotted as individual points.

It should be noted that these boxplots do not convey all the data needed for a complete assessment of BMP performance. In these plots the comparison is based exclusively on concentration achieved and ignores the additional load reduction that results from incidental infiltration in many types of BMPs. Data from the Los Angeles area indicates nearly 50% of the runoff may be lost to infiltration and evaporation in swales and somewhat smaller proportions in buffer strips and unlined extended detention basins. Where the emphasis on load reduction is paramount the impact of infiltration in BMPs should not be overlooked.

The standard parameters represent the following:

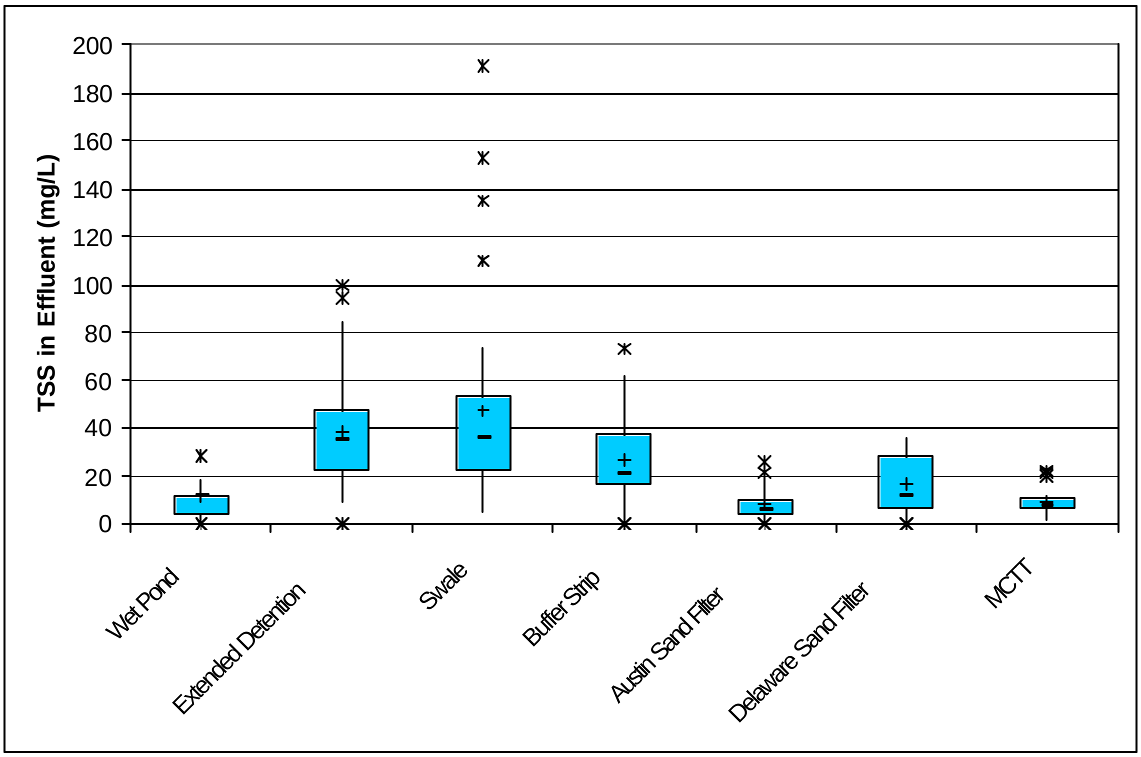



The box portion (75th Percentile – 25th Percentile) is the interquartile range or midrange. The optional box plot parameters are defined as follows:

| | |
|---|---|
| Upper Outer Fence | = 75th Percentile + 3.0(Midrange) |
| Upper Inner Fence | = 75th Percentile + 1.5(Midrange) |
| Upper End of Whisker | = Highest data point below Upper Inner Fence |
| Lower End of Whisker | = Lowest data point above Lower Inner Fence |
| Lower Inner Fence | = 25th Percentile – 1.5(Midrange) |
| Lower Outer Fence | = 25th Percentile – 3.0(Midrange) |

Figure E1-44 compares the observed effluent concentration for TSS for each of the BMPs. Figure E1-44 demonstrates the comparatively low TSS effluent concentrations produced by the sand filters (Delaware, Austin, and MCTT) and the wet basin. The small boxes for these devices are a testament to the consistent effluent created. Despite the relatively higher effluent concentrations for extended detention basins, and the vegetated controls, the overall difference in TSS load reduction, when the effects of infiltration within the BMPs is accounted for, is not large, with total reduction being about 80% or more for all the devices.

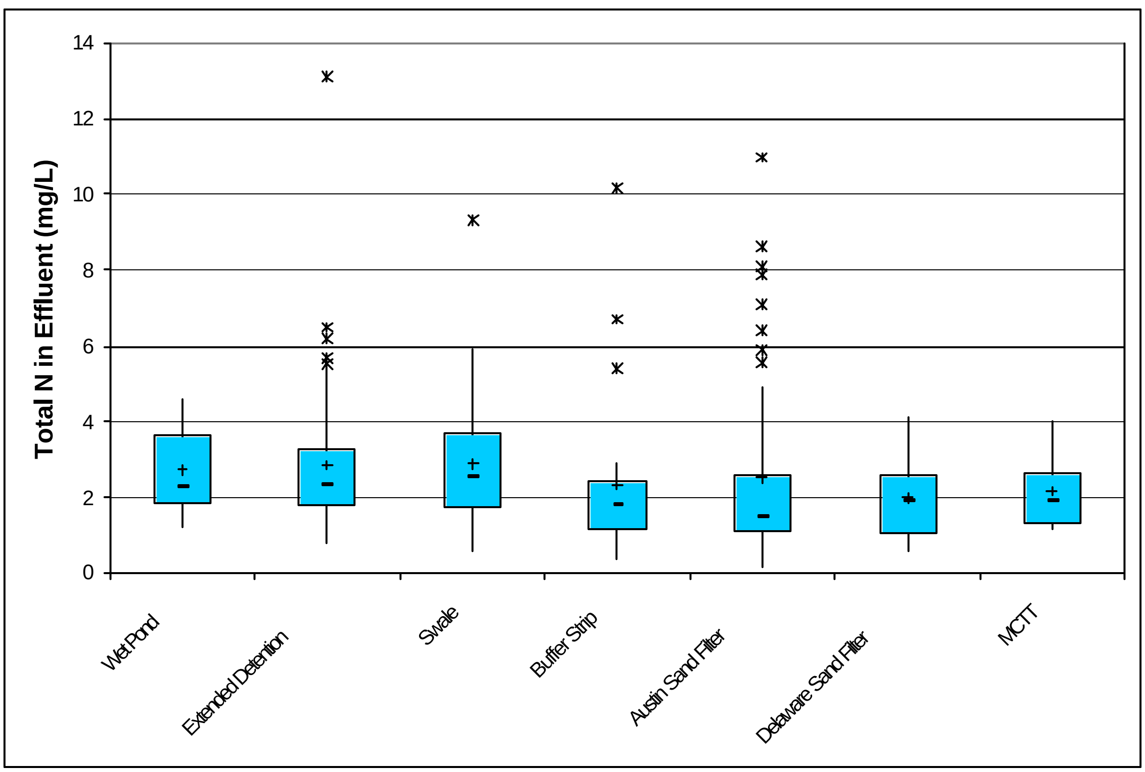
**Figure E1-44 Comparison of TSS Effluent Concentrations**




This comparison of TSS effluent concentrations contrasts dramatically with the results one gets by calculating the effectiveness using the efficiency ratio method, where the percent reduction using the same data set is 90% for Austin filters, only 75% for the MCTTs (which are functionally almost indistinguishable from the Austin design), and 76% for extended detention basins. Figure E1-44 demonstrates that the effluent quality of the MCTT and Austin filters is indistinguishable and far superior to that produced by extended detention basins. It is only because the MCTTs were located in a watershed with relatively low TSS influent concentrations that their performance relative to the other devices appears artificially depressed when using a conventional measure of performance.

The expected performance of the various BMPs for total nitrogen removal is compared in Figure E1-45. This figure shows that the effluent concentration for nitrogen is very similar for all of these BMPs. Part of the reason for this is that a substantial portion of the nitrogen is in soluble form (nitrate and ammonia). Wet ponds do not appear to perform substantially better than other BMPs even though they are often recommended for implementation when removal of dissolved constituents is an objective. One reason for this is that most of the water discharged from the wet pond during smaller events is displaced permanent pool. The concentration of the permanent pool is highly dependent upon the quality of the baseflow that enters the pond between events and the transformations that occur to that flow in the basin such as nutrient release from bottom sediments in response to equilibrium chemistry. At the site in the study, the dry weather influent nitrate concentration was almost 16 mg/L, which is substantially reduced during its residence time in the wet pond. Consequently, when nutrient reduction is estimated on an annual basis, including both dry and wet weather flows, the reduction is substantial.

**Figure E1-45 Comparison of Total Nitrogen Effluent Concentrations**



Observed effluent concentrations for total phosphorus are presented Figure E1-46. Swales, strips, and the wet basin all exhibit much higher effluent than influent concentrations. For the swales and strips, this may be related to leaching of nutrients from the dormant vegetation. The vegetation at these sites was a monoculture of salt grass (*Distichlis spicata*), which is a warm season grass that was dormant during the wet season. Other data indicate that phosphorus was leached from the dormant grass during storms, providing a relatively constant source.

The effluent quality of the wet basin is related primarily to the quality of the dry weather flow that is displaced from the permanent pool during storms and is independent of influent concentration as noted earlier. Consequently, these data should be used with care in estimating the performance of a wet basin relative to other BMPs if implemented at a site with higher quality perennial flow.

**Figure E1-46 Comparison of Total Phosphorus Effluent Concentrations**

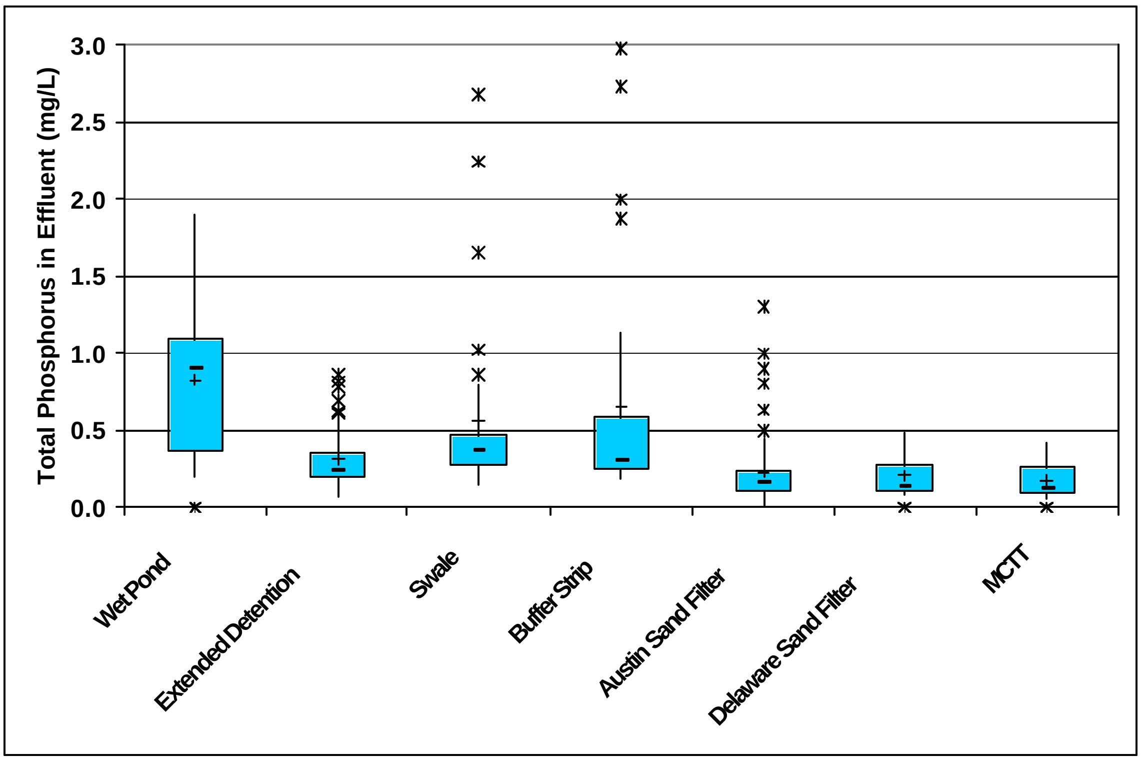



Figure E1-47 and Figure E1-48 compare the removal of dissolved copper and zinc for the subject BMPs. For these constituents, the load reductions associated with the swales and strips is comparable to other technologies. These results support the conclusions of Barrett et al. (1998), who reported that vegetated controls can be as effective as more complex systems for treating urban runoff.

Figure E1-49 compares the observed concentrations of fecal coliform in the effluent of the subject BMPs. Although substantial reduction is observed for many of the BMPs, contact recreation standards are only met in the discharge from the wet pond. This superior performance confirms the results presented for the wetlands constructed in Laguna Niguel.

**Figure E1-47 Comparison of Dissolved Copper Effluent Concentrations**

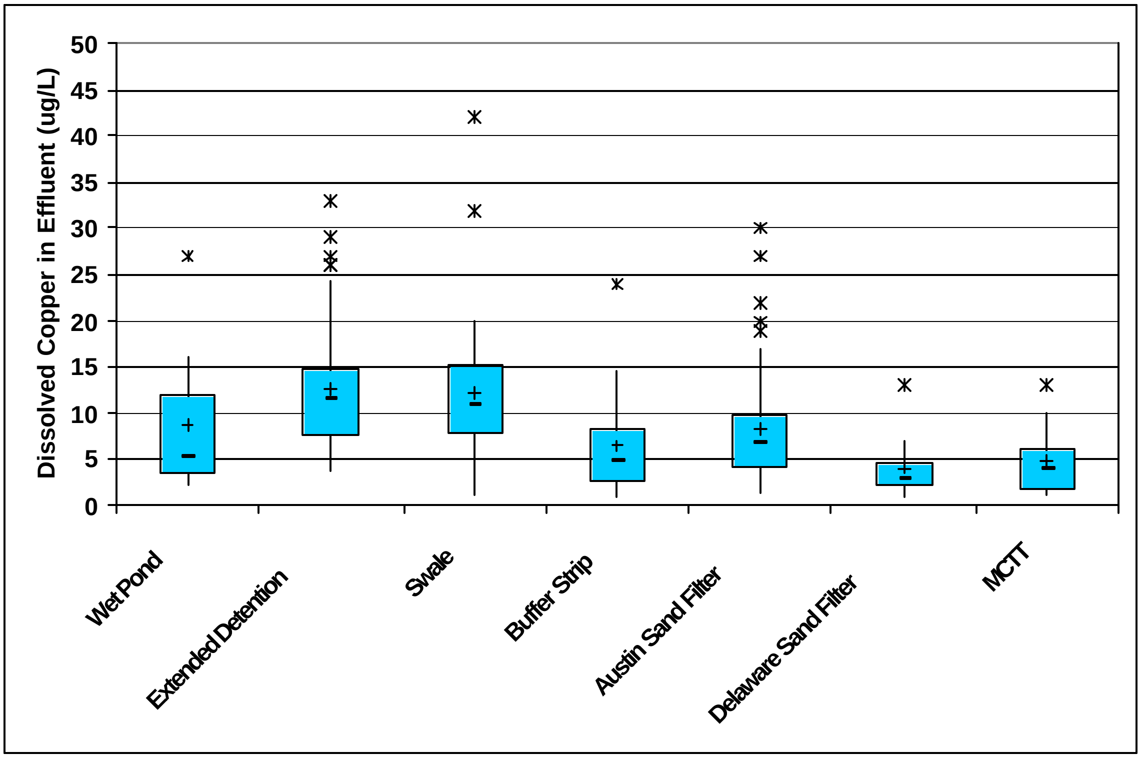


**Figure E1-48 Comparison of Dissolved Zn Effluent Concentrations**

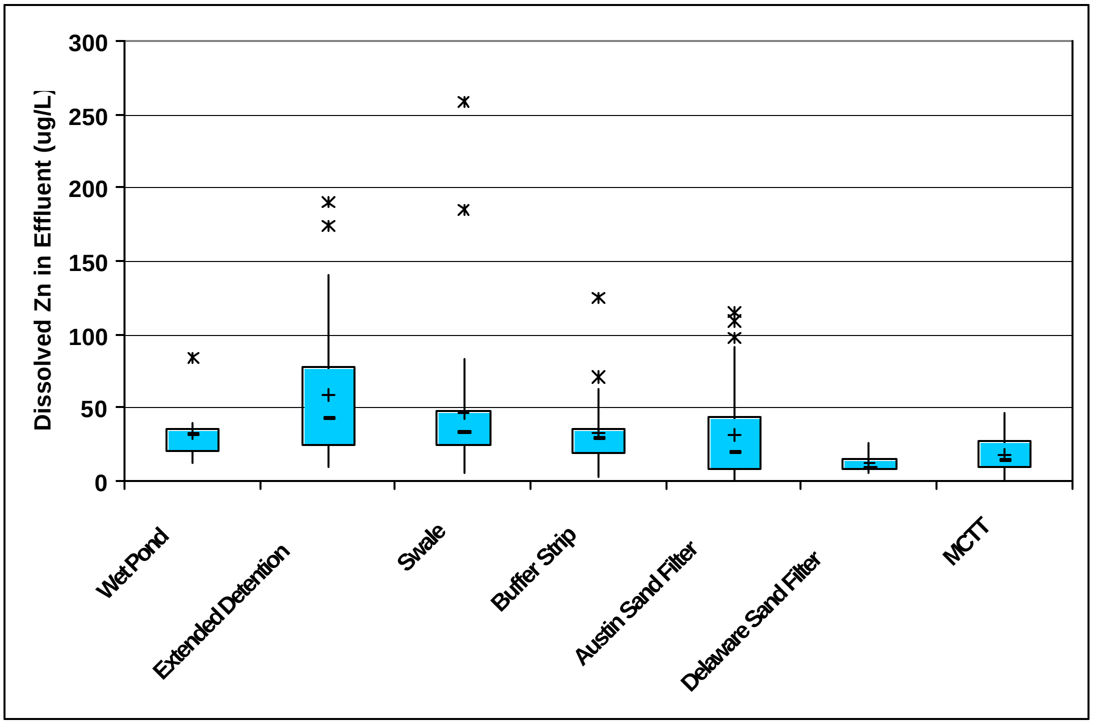
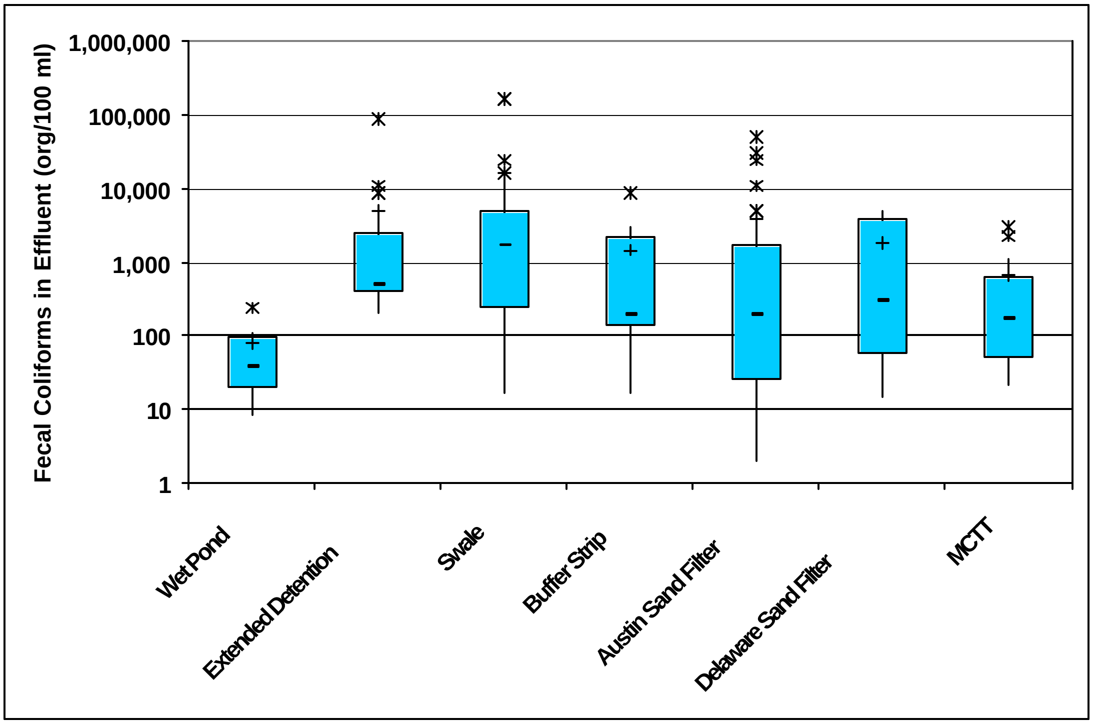



**Figure E1-49 Comparison of Fecal Coliform Effluent Concentrations**



### E1-4.2 Non-Chemical Assessments of Performance

Many times when BMPs are implemented at a site, regulatory agencies and other interested parties often want some proof that the BMP is, in fact, removing the constituents of concern. Chemical monitoring using automated water quality samples is an extremely expensive way and time consuming way to get this confirmation. Certainly, if a BMP is constructed similarly to other monitored BMPs, then the performance should be roughly the same. Over time there may be concern that insufficient maintenance has been performed to keep the facility operating at the design level; consequently, there needs to be a simple way to assess whether the expected performance is being realized. Therefore, the following paragraphs describe ways to assess performance of BMPs empirically without the necessity of chemical monitoring.

Qualitative monitoring provides a means of studying BMPs in a time and cost-effective manner. The approach is not aimed at identifying specific pollutant levels at unique points within the system, but at evaluating the status of the system, and the receiving watercourse. With appropriate tools, this evaluation can be undertaken consistently between a wide range of sites and should provide important details on:

- The change in hydraulic performance of the BMP system with time
- The influence of maintenance activities on performance
- The influence of particular design characteristics on performance
- The impact of the system on receiving watercourse quality
- The ecological status of the system
- The amenity and aesthetic benefits of the systems
- Any risks associated with the systems
- Likely appropriate operation and maintenance schedules

Because of differences in the design features, physical and chemical pollutant removal processes, and other factors it will be necessary to record different qualitative observations at each of the monitored BMPs. Some of the relevant questions that can be answered based on empirical observation for selected BMPs are shown below.

*Extended detention basins* – Does the basin drain from basin full conditions in the design drain time? Does it drain completely with no standing water? What is the performance of various outlet configurations and their ability to resist clogging? Do energy dissipaters prevent resuspension of material near the inlet? Can we observe whether geometry or other factors reduce/eliminate short circuiting? What is the required maintenance frequency? To answer these questions the following characteristics, among others, would be recorded:

- Depth of water in the basin
- Time since last storm event
- Total rainfall of last storm event
- Time to completely drain from basin full conditions
- Persistence of small pools or other saturated areas days after rainfall event
- Height and density of vegetation on the basin invert
- Evidence of outlet clogging
- Location and amount of accumulated sediment
- Evidence of sediment resuspension near inlet
- Type and amount of trash and debris removed during maintenance
- Evidence of erosion, scour, and slope instability

*Sand Filters* – Does the facility drain fully within the design time? How does the filter sizing criterion affect the maintenance interval? Can we relate variations in clogging to filter media grain size? Relevant observations would include:

- Depth of water in the basin
- Time since last storm event
- Total rainfall of last storm event
- Time to completely drain from basin full conditions
- Persistence of small pools or other saturated areas days after rainfall event
- Evidence of solids accumulation on the filter surface
- Depth of penetration of solids into the filter bed
- Height and density of vegetation on the basin invert
- Evidence of filter clogging
- Location and amount of accumulated sediment
- Evidence of sediment resuspension near inlet
- Type and amount of trash and debris removed during maintenance
- Evidence of erosion, scour, and slope instability

*Infiltration Devices* – Does the basin drain completely within the design drain time when new, document changes in drain time as facility ages – relate to soil / subsurface properties? Does presence of vegetation seem to help maintain initial infiltration rates?

- Depth of water in the basin
- Time since last storm event
- Total rainfall of last storm event

- Time to completely drain from basin full conditions
- Persistence of small pools or other saturated areas days after rainfall event
- Characteristics of the soil properties
- Depth to groundwater below the basin invert
- Height and density of vegetation on the basin invert
- Evidence of basin clogging
- Location and amount of accumulated sediment
- Type and amount of trash and debris removed during maintenance
- Evidence of erosion, scour, and slope instability

*Wet ponds* – Does vegetation establish rapidly and remain confined to designated areas such as the perimeter bench. Is there evidence that extensive vegetated cover promotes short circuiting? Is vegetation harvesting necessary for performance or nutrient removal?

- Depth of water in the basin
- Time since last storm event
- Total rainfall of last storm event
- Time to completely drain from basin full conditions
- Height and density of vegetation on the basin invert
- Distribution of emergent vegetation
- Evidence of short circuiting or dead areas due to vegetation growth
- Evidence of outlet clogging
- Location and amount of accumulated sediment
- Evidence of sediment resuspension near inlet
- Type and amount of trash and debris removed during maintenance
- Evidence of erosion, scour, and slope instability

*Swales* – Do current guidelines of slope and width lead to a stable cross section without channelization? Do we observe differences in stability related to mowing frequency or type of vegetation? Are nutrients exported during vegetation dormancy periods?

- Time since last storm event
- Total rainfall of last storm event
- Height and density of vegetation in the swale
- Date of last mowing
- Evidence of channelization
- Location and amount of accumulated sediment
- Evidence of sediment resuspension

- Type and amount of trash and debris removed during maintenance
- Evidence of erosion, scour, and slope instability

*Buffer strips* - Do current guidelines of slope and width lead to a stable situation? Do we observe differences in stability related to mowing frequency or type of vegetation? Export of nutrients during vegetation dormancy periods?

- Time since last storm event
- Total rainfall of last storm event
- Height and density of vegetation in the swale
- Date of last mowing
- Evidence of channelization
- Location and amount of accumulated sediment
- Evidence of sediment resuspension
- Type and amount of trash and debris removed during maintenance
- Evidence of erosion, scour, and slope instability

In addition to qualitative monitoring of individual facilities, one can include an evaluation of the streams and drainage courses downstream of these facilities. Factors that can be evaluated include evidence of erosion / avulsion, sedimentation, observation of benthic invertebrates or other fauna, and excessive periphytic algal growth. Unfortunately, historical studies designed specifically to determine the effect of BMPs on downstream biological communities have not generally been able to distinguish between watersheds with and without BMPs (May and Horner, 2002).

### E1-4.3 References

Barrett, M. E., P.M. Walsh, J.F. Malina, Jr., and R.J. Charbeneau, 1998. Performance of Vegetative Controls for Treating Highway Runoff. *ASCE Journal of Environmental Engineering*, vol. 124, No. 11, pp 1121-1128.

Caltrans. in press. *Final Report of the Retrofit Pilot Program*, California Department of Transportation, Report No. CTSW-RT-01-050.

Gilbert, R.O., 1987. *Statistical Methods for Environmental Pollution Monitoring*. Van Nostrand Reinhold, New York.

Glick, R., G. C. Chang, and M.E. Barrett, 1998. “Monitoring and Evaluation of Stormwater Quality Control Basins,” in *Watershed Management: Moving from Theory to Implementation*, Denver, CO, May 3-6, 1998, pp 369 – 376.

Horner, R.R. and C.R. Horner, 1999. "Performance of a Perimeter ("Delaware") Sand Filter," in Treating Stormwater Runoff from a Barge Loading Terminal. 1999," in *Proceedings of the Comprehensive Stormwater and Aquatic Ecosystem Management Conf.*, Auckland, N.Z., Feb., pp 183-192.

May, C., and Horner, R., 2002. "The Limitations of Mitigation-Based Stormwater Management in the Pacific Northwest and the Potential of a Conservation Strategy based on Low-Impact Development Principles," in *Proceedings of the Ninth International Conference on Urban Drainage,* Sept, 8-13, 2002, Portland, OR.

Martin, E.H. and Smoot, J.L., 1986. *Constituent-Load Changes in Urban Stormwater Runoff Routed Through a Detention Pond-Wetlands System in Central Florida,* USGS. Water-Resources Investigations Report 85-4310.

Pitt, R., B. Robertson, P. Barron, A. Ayyoubi, and S. Clark, 1999. *Stormwater Treatment at Critical Areas: The Multi-Chambered Treatment Train (MCTT).* U.S. Environmental Protection Agency, Wet Weather Flow Management Program, National Risk Management Research Laboratory. EPA/600/R-99/017. Cincinnati, Ohio.

RBF Consulting, 1998a. BMP *Retrofit Pilot Program Scoping Study, District 11*. California Department of Transportation, Report No. CTSW-RT-98-026.

RBF Consulting, 1998b. *BMP Retrofit Pilot Program Scoping Study, District 7,* California Department of Transportation, Report No. CTSW-RT-98-027.

Sansalone, J.J. and Glenn, D.W. , in press. "Physical and Chemical Characteristics of Urban Roadway Snow Residuals Generated from Traffic Activities," *Journal of Water, Air and Soil Pollution*.

Striker, E.W., M.M. Quigley, B.R. Urbonas, J.E. Jones, and J.K. Clary, 2001. "Determining Urban Storm Water BMP Effectiveness," *Journal of Water Resources Planning and Management,* Vol. 127, No. 3, pp 144-149.

Taylor, J.R., 1982. *An Introduction to Error Analysis: The Study of Uncertainties in Physical Measurements.* University Science Books, Mill Valley, CA.

Wonnacott, T.H., and R.J. Wonnacott, 1990. *Introductory Statistics: Fifth Edition,* John Wiley and Sons, New York.

## E1-5 BMP Recommendations and Selection Procedure

### E1-5.1 Local Needs

There are two primary drivers for the implementation of structural BMPs for urban runoff treatment in Orange County: 1) prevent degradation of receiving waters from new development in what are now undeveloped watersheds and 2) reduce the pollutant loads to receiving waters threatened by existing development and activities. Since the pollutants responsible for degradation, BMPs recommended for implementation for a specific site should reduce the concentrations and loads of these constituents of concern.

The adverse impacts of urbanization on biological communities, receiving water quality, and habitat are well known and have been summarized by many authors including Herricks (1995). These changes include higher concentrations of many potentially toxic constituents, degradation of channel bed and bank resulting in habitat loss, and loss of sensitive species. Implementing appropriate structural BMPs will improve discharge quality and reduce peak flow rates, helping to protect receiving waters.

Many of the watersheds in urban areas of Orange County have already been identified as not supporting desired uses, such as contact recreation. Table E1-49 presents a list of these waterbodies and the constituents identified as responsible for causing these impairments. Additional development in these watersheds will likely lead to further degradation unless the impacts of this development can be mitigated through the use of structural controls or other measures.

Indicator bacteria and pathogens are clearly one of most important constituents of concern. As discussed in Section 4.1, of the common public domain BMPs only wet ponds are consistently capable of meeting contact recreation standards for fecal coliform, nevertheless, substantial reduction is observed for many of the controls. In general, no reduction in bacteria concentration is observed for buffer strips and swales.

Metals also often contribute to degradation of receiving waters in Orange County. Many common conventional structural BMPs offer substantial removal of total recoverable metals, with some providing significant reduction in the dissolved phase as well. Substantial reduction in concentrations of nutrients, dissolved pesticides may be difficult to obtain with current technology.

Selection of the appropriate BMPs for use in Orange County are also subject to local factors such as climate, soil types, and regulatory drivers. The Mediterranean climate of southern California

## Table E1-49 List of Impaired Water Bodies in Orange County

| Region | Orange County Water Body | Pollutant: Indicator Bacteria | Pathogens | Metals | Nutrients | Pesticides | Toxicity | Trash | TDS | Turbidity |
|---|---|---|---|---|---|---|---|---|---|---|
| Region 8 Santa Ana | Buck Gully Creek | X | | | | | | | | |
| | Huntington Beach State Park | X | | | | | | | | |
| | Huntington Harbor | | X | | | | | | | |
| | Los Trancos Creek (Crystal Cove Creek) | X | | | | | | | | |
| | Newport Bay, Lower | | | X | | X | | | | |
| | Newport Bay, Upper (Ecological Reserve) | | | X | | X | | | | |
| | Orange County Beaches | | | | | | | X | | |
| | San Diego Creek, Reach 1 | X | | | | X | | | | |
| | San Diego Creek, Reach 2 | | | X | | | X | | | |
| | Seal Beach | X | | | | | | | | |
| | Silverado Creek | | X | | | | | | X | |
| Region 9 San Diego | Aliso Creek (Mouth) | X | | | | | | | | |
| | Aliso Creek | X | | | X | | X | | | |
| | Dana Point Harbor | X | | X | | | | | | |
| | Pacific Ocean Shoreline, Aliso Beach HSA | X | | | | | | | | |
| | Pacific Ocean Shoreline, Dana Point HSA | X | | | | | | | | |
| | Pacific Ocean Shoreline, Laguna Beach and San Joaquin Hills HSAs | X | | | | | | | | |
| | Pacific Ocean Shoreline, Lowe San Juan HSA | X | | | | | | | | |
| | Pacific Ocean Shoreline, San Clemente, San Mateo, and San Onofre HSAs | X | | | | | | | | |
| | Prima Deshecha Creek | | | | X | | | | | X |
| | San Juan Creek | X | | | | | | | | |
| | San Juan Creek (Mouth) | X | | | | | | | | |
| | Segunda Deshecha Creek | | | | X | | | | | X |

is characterized by distinct wet and dry seasons that can make implementation of wet ponds and vegetated controls more difficult. In addition, many parts of Orange County are underlain by soil types with low permeability, restricting the use of infiltration BMPs. Finally, requirements of regulatory agencies, such as NPDES permits, vector concerns, and endangered species, may impact the applicability of certain BMPs.

To help in the selection of the most appropriate BMP for a given site, a selection matrix has been developed to highlight the important considerations for each type of BMP. The factors that need to be considered include issues such as catchment area, soil type, space requirements, and level of treatment. Using this matrix, the list of potential BMPs for a given site can be can quickly be narrowed to just a handful based on the specific site conditions.

### E1-5.2 Climatic Conditions

Southern California differs from many other areas in the country where BMPs have been evaluated in that there are pronounced wet and dry seasons. The wet season occurs mainly from the beginning of November to about the end of April. Approximately 10-12 inches of rainfall occur during this period. The dry season is from May to October when it is not uncommon to record no measurable rainfall.

The pronounced difference in these two seasons has broad implications for the BMPs that might be selected for Orange County, particularly wet ponds and vegetated controls. For many watersheds there will not be sufficient dry weather flow during the summer to support a conventional wet pond. This is especially true for small, highly impervious watersheds. Consequently, it is unlikely that wet ponds will be one of the more common BMPs implemented for stormwater treatment.

On the other hand, small constructed wetlands have been shown to be viable for treatment of dry weather flows in the County (e.g., the wetlands in Laguna Niguel). Because of their small size, which is mainly a function of the space available and the amount of flow available to sustain the vegetation, they are unlikely to provide the same level of performance for wet weather flow treatment as BMPs sized to treat 85% of the annual runoff.

The lack of rainfall over the summer months and the relatively low total annual rainfall means that a high level of vegetation density may be difficult to sustain without supplemental irrigation. The Caltrans study (2002) found that even incomplete vegetation coverage in vegetated swales and buffer strips could still provide substantial water quality benefits. A related issue in the Orange County area is the abundance of gophers. These animals can be very destructive of the vegetation and expose large amounts of soil to erosion. The bottom line is that use of vegetated controls in areas not subject to frequent maintenance needs to be monitored more carefully to ensure that a sufficient level of vegetation is maintained to stabilize the treatment area, reduce runoff velocities, and remove pollutants.

The vegetation and gopher issues are much reduced in landscaped areas around residential, retail, and office developments. These areas are highly visible and an aesthetic amenity and, consequently, normally are sufficiently maintained so that they will be effective for improving the quality of stormwater runoff.

The other BMPs reviewed will operate successfully in the Southern California climate. In fact, the performance of infiltration devices may be improved by the long recovery period during the summer months when the soil can dry out and groundwater levels beneath the basins and trenches decline.

There are a number of advantages to having distinct wet and dry seasons. These include the potential for conjunctive use facilities such as off-line extended detention basins and vegetated controls which can provide aesthetic, landscape, and passive play benefits. In addition, the long dry period allows infiltration facilities to fully recover and groundwater levels below them to subside. Another advantage of the relatively dry climate is that total annual of loading of TSS and other constituents is much lower than that which occurs in wetter climates because the average concentration in runoff is roughly the same everywhere. Consequently, some types of maintenance activities, such as sediment removal, will not be required as often in Orange County as in other locales.

### E1-5.3 Soil/Subsurface Conditions

Soil type and groundwater elevation are important for determining whether infiltration BMPs are appropriate for a given site. As described in the literature review, Hydrological Soil Group A or B (sand or sandy loam) is required to provide sufficient permeability for successful implementation of infiltration basins and trenches. Soils with lower permeability clog easily and drain so slowly that very large infiltration areas are required to prevent runoff from standing for more than 72 hours in the facilities.

Similarly, as runoff infiltrates the water table below the facility rises. Consequently, if there is not sufficient separation between the bottom of the facility and the groundwater elevation, a groundwater mound will develop forcing the infiltrated runoff to move laterally rather than down. This greatly reduces the infiltration rate and again can result in standing water in the device.

Figure E1-50 presents a map of the distribution of soil types in Orange County. Hydrological Group A, which represents the best chance for successful implementation of infiltration devices, is common in the central County north of the Santa Anna River. Group B soils are also widespread in the central area south of the Santa Anna River. Group C and D soils, which have higher clay content are concentrated in the highland areas.

In addition to appropriate soil type, it is also necessary to establish that the groundwater surface lies at least 10 feet below the invert of any proposed facility. Groundwater elevations have a wide variation, so a site-specific determination of the elevation at any particular site would need to be performed before an infiltration BMP is selected.

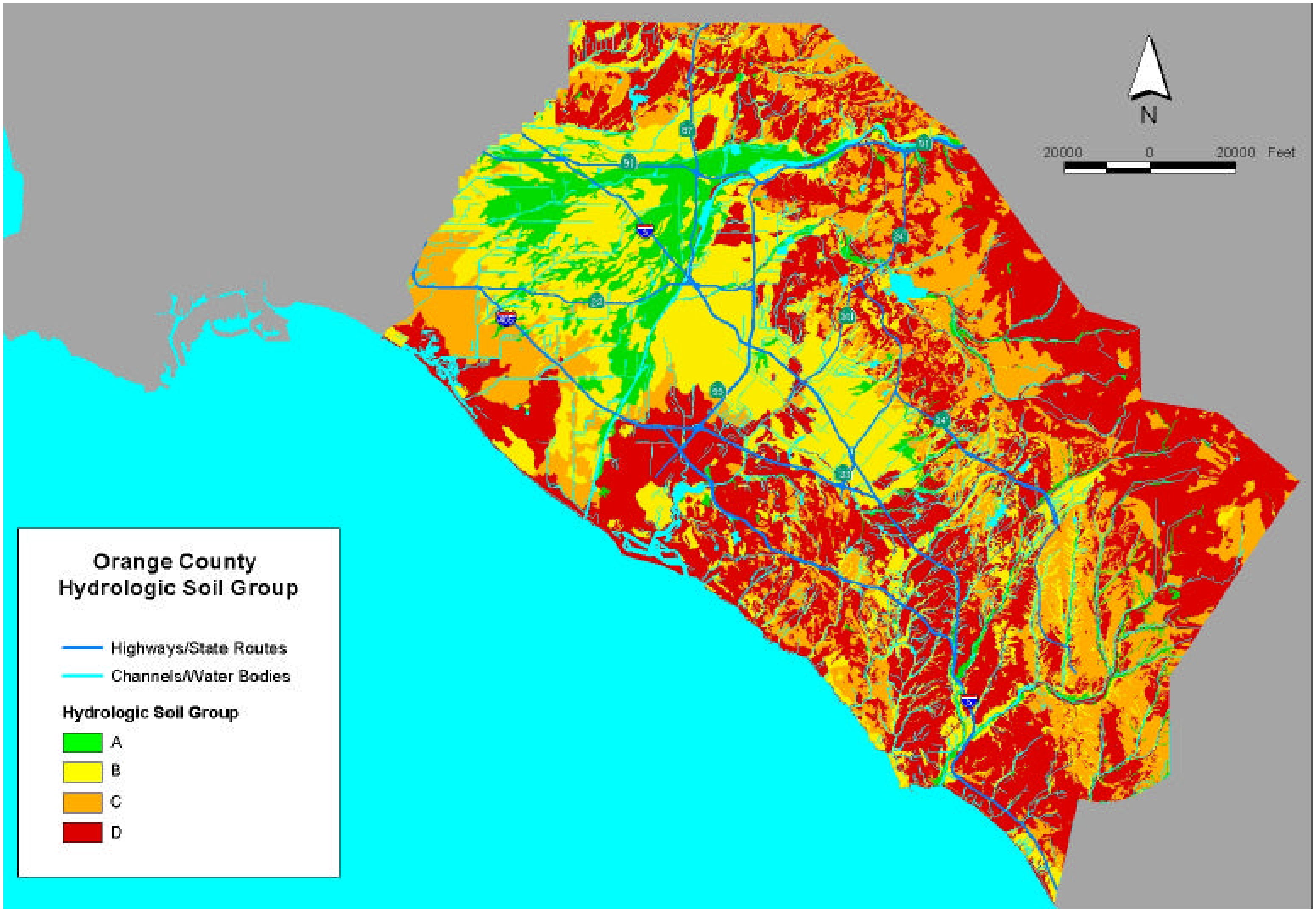



**Figure E1-50 Soil Types for Orange County**

## E1-5.4 Regulatory Issues

### E1-5.4.1 NPDES Permits

Current NPDES permits issued to Orange County by the Santa Ana and San Diego Regional Water Quality Boards have specific requirements for implementation of structural BMPs for new developments and significant redevelopment, which are included in Appendix A. These requirements include the following sizing criteria:

A. Volume

Volume–based BMPs shall be designed to infiltrate, filter, or treat either:

1) The volume of runoff produced from a 24-hour, 85th percentile storm event, as determined from the local historical rainfall record; or,
2) The volume of annual runoff produced by the 85th percentile, 24-hour rainfall event, determined as the maximized capture stormwater volume for the area, from the formula recommended in Urban Runoff Quality Management, WEF Manual of Practice No. 23 / ASCE Manual of Practice No. 87 (1998); or,
3) The volume of annual runoff based on unit basin storage volume, to achieve 80% or more volume treatment by the method recommended in California Stormwater Best Management Practices Handbook – Industrial / Commercial (1993); or,
4) The volume of runoff, as determined from the local historical rainfall record, that achieves approximately the same reduction in pollutant loads and flows as achieved by mitigation of the 85th percentile, 24-hour runoff event;

OR

B. Flow

Flow-based BMPs shall be designed to infiltrate, filter, or treat either:

1) The maximum flow rate of runoff produced from a rainfall intensity of 0.2 inch of rainfall per hour; or,
2) The maximum flow rate of runoff produced by the 85th percentile hourly rainfall intensity, as determined from the local historical rainfall record, multiplied by a factor of two; or,
3) The maximum flow rate of runoff, as determined from the local historical rainfall record,

that achieves approximately the same reduction in pollutant loads and flows as achieved by mitigation of the 85th percentile hourly rainfall intensity multiplied by a factor of two.

#### E1-5.4.2 DHS and Vector Management

BMPs can create permanent or semi-permanent standing water and may present opportunities for vectors to establish themselves and potentially spread disease among the general public. It is clear from the Caltrans BMP retrofit program that existing stormwater drainage systems may be modified by adding devices that may contain a permanent pool. Such retrofitting may create opportunities for disease carrying or nuisance organisms to flourish, especially organisms such as mosquitoes that may have to be controlled. Within the loose framework provided by the applicable public health statutes, the BMPs may be viewed as “threats to public health.” In this situation, several organisms may pose a threat to public health and comfort:

- Mosquitoes are the vectors of primary concern and are the most prominent of the numerous bloodsucking organisms that attack humans. They are the only known means of transmission of the causal agents of several diseases that, in the recent past, have been common in the United States. They require standing water for development, though adults may travel far from such sites in search of a blood meal.

- Black flies are biting flies that inflict a painful bite. Like mosquitoes, only the female takes blood meals. Many humans react adversely to black fly bites. While black flies are important vectors of human disease in the New World and Old World tropics, they do not transmit any known disease to humans living in California. The immature stages of black flies are found in flowing water where the larvae form aggregations on stones, vegetation, debris, and other objects in flowing water. Pupae can be found attached to the larval substrates.

- Cockroaches are considered synanthropic species adapted to living in close association with humans. As such, cockroaches may be encountered in some BMPs that provide habitat and coverage for them. They have never been considered an important vector of disease but are considered a nuisance. The local vector control districts (VCDs) originally requested that Caltrans monitor for these organisms in the BMPs. However, after further consultation, it was determined that these organisms posed little threat to the general population and did not merit inclusion in the general monitoring scheme.

- Midges are small flies that are often mistaken for mosquitoes. Although midges do not feed on blood like mosquitoes, they can become a serious nuisance near wetlands, drainage channels, lakes, golf course ponds, etc. The immature stages of the midge life cycle occur in water. Adult midges are very similar in appearance to mosquitoes and are attracted to lights and to vertical structures, such as the walls of human residences, adjacent to larval habitats. Midges can be a serious concern near airports where swarms of the flies are attracted to runway lights and can interfere with airport operations. These organisms are included in the monitoring program as both adults and larvae. However, larval midge monitoring is not initiated until sizeable numbers are collected in adult traps.

- Vertebrates that are considered “vectors” are actually “reservoirs” or hosts, and the agents that cause human disease are transmitted by their arthropod parasites. A classic example is plague. The plague bacterium, *Yersinia pestis*, is transmitted to humans by fleas that live on rodents. In California, ground squirrels, chipmunks, and wood rats have also been implicated in the transmission of plague. Other human diseases associated with vertebrates and their parasites are murine typhus, relapsing fever, and Lyme disease. Animals can be true vectors and transmit disease through bites or other close contact. Rabies, for instance, is transmitted when infected saliva enters through a break in the skin. In California, skunks, bats, and foxes are most commonly associated with rabies. Hanta virus pulmonary syndrome is transmitted directly by humans inhaling the virus in feces particles or the urine of deer mice. Other diseases associated with vertebrate vectors in California include tularemia, Rocky Mountain spotted fever, and leptospirosis. Vertebrate organisms of concern for this project are rodents (rats, mice and squirrels), skunks and opossums.

The laws and regulations that govern or relate to mosquito and vector control in California are found principally in the sections of the California Health and Safety Code, Civil Code, Food and Agricultural Code. Health and Safety Code Sections 2270-2294 describe “District Powers.” The Public Health and Safety Code has legal precedence over many other regulations. Legal opinions regarding issues relating to priority of enforcement for Public Health and Safety Code Sections 2200 and 2292 versus other statutes determined that, with adequate notice, vector control agents had enforcement priority and that other agencies could be held criminally liable for interference with vector control efforts.

The health code statutes, as written, give vector control district managers wide latitude in determining what constitutes a public health threat. If these statutes are interpreted narrowly, it is conceivable that the mere presence of “open, standing water” could be construed as a threat to public health, and may be abated accordingly. As such, only *prima facie* evidence of breeding (i.e., the presence of only one mosquito larva) is required for abatement. Under these conditions, it is the vector control district managers who largely determine under what conditions abatement will occur. The vector control districts in Los Angeles County have established an abatement threshold of one larva for the BMPs. With this threshold, these districts can abate when a single larva is collected from a site. The San Diego County Vector Surveillance and Control Division generally does not rely on arbitrary thresholds in determining abatement needs. San Diego County prefers an approach where factors such as BMP location, larval density, and proximity to residential areas are considered.

In order to reduce the production of vectors and nuisance insects and minimize the application of pesticides at BMP sites, effective habitat management is essential. Management strategies should focus on three general areas: physical control, biological control, and chemical control.

Physical Control - Water management plays a major role in controlling vectors. Standing water should not persist more than three days (72 hours) in any BMPs except for wet basins that require a permanent pool. Even habitat that is only temporarily inundated is capable of supporting mosquito production. Control of aquatic vegetation is a critical component of controlling vectors in permanent standing water. Vegetation provides refuge from predation and physical disturbance (e.g., waves), and food resources for mosquitoes. Vegetation management also aids in the control of mosquitoes. Standing water that contains vegetation or organic debris is also a problem as they provide ideal egg-laying habitat.

Biological Control - The biological control agent most commonly used to control mosquitoes is the mosquito fish, *Gambusia affinis*. Mosquito fish are most effective in wet basins that have a depth of 4 to 12 feet and limited shallow shoreline (less than 30 percent of surface area). Their effectiveness as a mosquito control agent declines greatly as the density of vegetation increases.

Chemical Control - Vector control agencies prefer the employment of new generation biologically rational compounds for mosquito control. These compounds pose an insignificant threat to non-target organisms and are considered the most environmentally friendly approach to mosquito control. Most vector control districts focus on controlling mosquito larvae with compounds such as mosquito-specific bacteria in the genus *Bacillus* and insect growth regulators (Ergs). Control of mosquitoes in the adult stage is rare, and generally occurs only as a last resort. In Southern California, three types of pesticides are currently used against mosquito larvae: mosquitocidal oils, mosquito-specific bacteria, and insect growth regulators (IGRs).

- Mosquitocidal oil kills mosquito larvae and pupae by suffocation. This oil is the only material available for use in California that is effective against mosquito pupae. The oil dissipates within 48 to 72 hours after application. Mosquitocidal oils are used when pupae are present, or when the conditions are such that other pesticides will be ineffective.

- Mosquito-specific bacterial pesticides are widely used throughout the world for mosquito control. *Bacillus thuringiensis* variety *israelensis* (*Bti*) and *Bacillus sphaericus* are currently registered for use in California. Both *Bti* and *B. sphaericus* are harmless to humans and non-target organisms when applied according to the label directions. The toxins they produce are specific to mosquitoes and degrade rapidly in aquatic environments. The application of *Bti* and *B. sphaericus* to BMPs should not compromise the water quality.
- Insect growth regulators (IGRs) are synthetic hormones that inhibit the development of mosquitoes. When applied to sites where larvae are present, mosquitoes fail to develop into adults. IGRs are available in several formulations including liquid concentrates, pellets, and briquettes, and can also be applied in combination with Bti. Methoprene (Altosid) is the most common IGR currently in use with vector control districts in southern California. Liquid formulations of methoprene are currently being used as a regularly part of an integrated control program against mosquitoes in Los Angeles County. Briquette and granule formulations are also available. Briquettes are capable of releasing methoprene over a 30 to 120 day period.

Eight mosquito species were collected from Caltrans BMP structures in Los Angeles and San Diego during the 2-year study, four of which are known vectors of human disease. Of the nine different BMP technologies implemented by Caltrans, those that maintained permanent sources of standing water (i.e. Multi-Chambered Treatment Trains (MCTT), Continuous Deflective Separators (CDS), and wet basins) provided excellent habitat for immature mosquitoes, and frequently supported large populations relative to other structural designs. In contrast, BMPs designed to drain rapidly (i.e. biofiltration swales and strips, sand media filters, infiltration basins and trenches, drain inlet inserts, extended detention basins) provided less suitable habitats for vectors.

Results showed that vector production at the Caltrans BMP structures was influenced not only by design, but also by factors such as location, immediate and large-scale surroundings, non-stormwater discharges (e.g. irrigation), site maintenance, and various other unexpected events. Because of this, direct comparisons between structures of similar design were difficult, if not impossible. BMP design features identified during this study that contributed to vector production should be avoided in future construction plans. Some examples include the use of any sump, catch basin, or spreader trough that does not drain down completely, loose rip rap, pumps or motors that "automatically" drain water, and effluent pipes with small diameter metering holes that may be prone to clogging. If absolutely necessary, sumps should be covered by a suitable mosquito net that is inspected and replaced on a regular basis.

Permanent ponds will always be the source of some vectors regardless of design. To minimize vector production, ponds should be stocked with mosquito fish and be constructed with steep banks to reduce potential breeding habitats (although steep banks may hinder the ponds pollutant removal effectiveness and are a safety concern). Although shallow pond grades and emergent vegetation increase water quality benefits, steep banks would reduce potential vector sources and vegetation density, benefiting long-term water quality and vector control.

Fourteen recommendations of the vector control districts regarding BMP implementation are summarized below:

- One of the more effective environmental controls for vegetation, and consequently mosquitoes, is proper water management. Standing water that contains vegetation or organic debris is typically a problem, but habitats that are temporarily inundated can also create mosquito problems.
- Vector control strategies should concentrate on physical measures, minimizing the amount standing water present in the devices. Standing water that persists for three days (72 hours) or longer, especially during warm periods, is likely to produce adult mosquitoes.
- Access to some BMPs will be provided through manholes or grates; vectors will readily enter and exit the structures. Any access cover should be free of apertures large enough to allow entry of adult mosquitoes if a permanent pool of water is maintained in the structure.
- Mosquito production is strongly associated with vegetation densities and coverage. BMP designs that support reduced vegetation growth are preferable for the enhancement of vector control. From a vector control viewpoint, the negative attributes of thick vegetation outweigh many of the benefits. Thick vegetation and inadequate access hinder mosquito abatement. In addition to providing habitat for mosquito larvae and resting sites for adult mosquitoes, thick vegetation will inhibit the application of larvicides.
- If a given site does not drain, heavy equipment used for vegetation maintenance might compromise the integrity of the site by channeling water flow and creating additional habitats (e.g., tire tracks, unevenness in the basin floor that would lead to standing water) for vectors of disease. Scheduling vegetation maintenance as near as possible to the onset of mosquito season (early April) is advisable. Doing so will limit the vegetation in the basin to the greatest extent possible once mosquito populations are likely to increase in the spring. Even though climatic conditions are potentially favorable for year-round growth, some senescence and slowed growth of the vegetation will occur

during the winter months. If necessary because of basin characteristics or permit considerations, the vegetation management could be carried out in the late autumn or early winter.

- Vegetation removed from the site will need to be transported elsewhere for disposal. Allowing downed vegetation to remain on site and then inundated may create a severe vector problem.
- Infiltration ponds may be an even greater vector control and vegetation management problem than are detention basins because they encourage the development and accumulation of vegetative detritus that upon re-inundation is attractive to egg-laying mosquitoes and provides resources for mosquito larvae. These sites encourage the growth of vegetation, yet infiltration considerations necessarily limit vegetation management practices.
- Biofiltration swales and strips are intended to serve as filter-conduits to move water from impervious surfaces to other holding structures or the receiving water. As long as standing water does not occur in the swales, the slopes of the designs and the short water residence time should prevent a vector problem. Bacterial degradation of grass 16 clippings can be highly attractive to gravid (egg-carrying) *Culex tarsalis* and *Cx. quinquefasciatus*.
- If the site does not dry, heavy equipment used for vegetation maintenance might compromise the integrity of the site by channeling water flow and creating additional habitats (e.g., tire tracks, unevenness in the basin floor that would lead to standing water) for vectors of disease. Efforts to maintain the integrity of the basin floor should be maximized.
- Scheduling vegetation maintenance as near as possible to the onset of mosquito season (early April) is advisable. Doing so will limit the vegetation in the basin to the greatest extent possible once mosquito populations are likely to increase in the spring. Even though climatic conditions are potentially favorable for year-round growth, some senescence and slowed growth of the vegetation will occur during the winter months. If necessary, because of basin characteristics or permit considerations, the vegetation management could be carried out in the late autumn or early winter. Vegetation removed from the site will need to be transported elsewhere for disposal. Allowing downed vegetation to remain on site and then inundated will create a severe vector problem.

**General Design and Maintenance Recommendations**

Mosquitoes were found to be the most significant and persistent vectors associated with BMP structures that retained a permanent pool of water between storm periods. Because of this, recommendations focused primarily on preventing standing water, which is needed for the development of immature mosquitoes. As a general recommendation, all BMP structures should be easily and safely accessible (e.g. avoid structures with confined space) to allow vector control personnel to effectively monitor and, if necessary, abate vectors.

As a general rule, mosquito larvicides are applied with hand-held equipment at sites with small footprints and with backpack or truck-mounted high-pressure sprayers at sites with large footprints. The effective swath width of most backpack or truck-mounted larvicide sprayers (liquid or granule) is approximately seven meters on a windless day. As a result, road access (with provisions for turning a full-size work vehicle) should be provided along at least one side of large, open BMPs that are less than seven meters wide.

Those BMPs that have shoreline-to shoreline distances in excess of seven meters should have a perimeter road for access to both sides. It is also important that no vegetation or other obstructions be present between the access road and the BMP, which might obstruct the path of larvicides to the water. Thus, roads should be built as close to the shoreline as possible. The periodic removal or mowing of invasive cattails or other tall wetland vegetation including shrubs and trees is necessary. The following criteria should be incorporated into the design of all structural BMPs to reduce the probability of mosquito breeding.

*Dry Systems*

- Structures should be designed such that they do not hold standing water for more than 72 hours (the minimum length of time for mosquito development). Provisions to prevent or reduce the possibility of clogged discharge orifices (e.g. debris screens) should be incorporated into the design. The use of weep holes are not recommended due to rapid clogging when adjacent to or within a sediment laden area.
- The hydraulic grade line of each site should be a primary factor in determining the appropriate BMP that will allow water to flow by gravity through the structure (Pumps are subject to failure and require sumps that hold water, which may create mosquito habitats. Structures that do not require pumping should be favored over those that do).
- Designs should avoid the use of riprap or concrete depressions that may hold standing water.
- Distribution piping and containment basins should be designed with adequate slopes to

drain fully and prevent standing water.

*Systems with Sumps, Basins or Permanent Ponds*

Structures designed with sumps or basins that retain water permanently or semi-permanently (e.g. MCTT, CDS, Delaware-type sand media filters, canister-type media filters) should be sealed completely against adult mosquitoes. Adult female mosquitoes may utilize openings as small as 1/16th of an inch to access water for egg-laying.

- Structures should be designed with the appropriate pumping, piping, valves, or other necessary equipment to allow for easy dewatering of the unit if necessary.
- If the sump or basin is completely sealed, with the exception of the inlet and outlet, the inlet and outlet should be fully submerged so that female mosquitoes have access to only a limited surface area of water for egg-laying.
- Permanent ponds should maintain water quality and quantity sufficient to support surface-feeding fish such as mosquito fish, *Gambusia affinis*, which feed on mosquito larvae.
- Permanent pond shorelines should be accessible to vector control crews for routine monitoring and abatement procedures, if necessary. Emergent plant density should be controlled so that natural mosquito predators are not inhibited or excluded from pond edges (i.e. fish should be able to swim between plant bases).
- If possible, permanent ponds should be maintained with depths in excess of four feet to preclude invasive emergent vegetation such as cattails. Pond edges below the water surface should be fairly steep and uniform to discourage dense plant growth and reduce favorable mosquito habitat.
- Riprap or liners should be used in areas where vegetation is not necessary, to prevent unwanted growth.
- Permanent ponds should be designed to allow for easy dewatering of the basin when needed.

An unexpected element at the beginning of the Caltrans study concerning the feasibility of the devices was the importance of avoiding standing water in the BMPs. Standing water presents opportunities for vectors to establish themselves, and mosquito breeding was observed at all of the sites where standing water persisted for at least 72 hr. In addition to the technologies that incorporate a permanent pool (i.e., wet basin, MCTT, StormFilter™ and Delaware filter), standing water also occurred in stilling basins, around riprap used for energy dissipation, in flow spreaders, and in some outlet structures. To minimize vector concerns in any future installations, these nonessential pools should be avoided during design.

#### E1-5.4.3 USFWS Endangered Species Issues

Biological issues were an important concern for BMP operation and maintenance. The presence of endangered species, threatened species and species of special concern in a BMP could affect scheduled maintenance and other activities. Early and effective coordination with the U.S. Fish and Wildlife Service (USFWS) and the California Department of Fish and Game could avert some of the problems associated with the presence of biological resources; however, the potential presence of protected species may result in siting, construction, operation, and maintenance restrictions.

In San Diego County there were several species of concern that were monitored for during the Caltrans study. The nesting period of the least tern was a concern at the La Costa Austin filter and construction had to be delayed until the end of the nesting period. Nets were installed over the Austin sand filters and infiltration basin during the dry season to prevent the nesting of the least tern and Snowy Plover in the sand filter bed. Mylar strips were used at the La Costa wet basin to discourage the nesting of sensitive species in the wet pond vegetation. Salt grass used in biofilters is also a habitat for the salt marsh skipper (butterfly). The sites in San Diego were monitored for the presence of the skipper. In Los Angeles, the primary concern was the opportunity for burrowing owls, an endangered species, to use the gopher mounds and ground squirrel burrows. There was abundant gopher activity at many of the swales and detention basins in the area, but no owls were observed.

#### E1-5.4.4 Coastal Commission

The California Coastal Commission has authority over many development issues in the coastal areas that may affect BMP selection and implementation. For instance, in the Caltrans study, trees located adjacent to a swale for the I-5/Palomar Airport Road offramp had to be protected in accordance with the Coastal Development Permit (CDP) in effect for the Cannon Road improvements. The CDP required that any existing trees that would be removed by construction activities be replaced at a 5:1 ratio. The BMP was redesigned to eliminate the need for mitigation by confining flow in concrete channels around the two areas of concern. To further protect the trees, excavation activities were restricted to the area beyond the tree dripline. Consequently, the BMP facility as constructed incorporates three biofiltration swales and two intermediate concrete swales.

### E1-5.5 Most Applicable BMPs

An important component of this report is to develop recommendations for the most applicable BMPs for Orange County. The applicability is based on a number of criteria identified through a comprehensive literature review. Factors affecting the inclusion of a BMP on this list include:

- Effectiveness of pollutant reduction
- History of successful implementation elsewhere
- Experience in Southern California with the same or similar BMPs
- Amount and type of maintenance

Based on these criteria, the following BMPs are recommended for consideration for implementation:

- Extended Detention Basins
- Vegetated Swales
- Vegetated Buffer Strips
- Bioretention
- Sand and Organic Filters
- Infiltration Basins
- Infiltration Trenches

These BMPs have been widely implemented and operated successfully in many areas including southern California. One must be particularly careful about the use of infiltration trenches and basins, because of potential issues related to groundwater contamination. In addition, clogging of these devices make be difficult to mitigate unless the underlying soils and groundwater levels are optimum for their use.

In the areas where dry weather flows are available for sustaining a perennial pool or the wetland vegetation, the following BMPs may also prove suitable.

- Wet ponds
- Constructed Wetlands

There are several important considerations when considering wet ponds and constructed wetlands. One is the potential benefit of these devices comes from treating dry weather flow from the contributing watershed. These flows often have high bacteria counts, which are substantially reduced in wet ponds and wetlands. In addition, open water is often viewed as an amenity by local property owners. On the other hand, vector considerations may require substantial annual maintenance, making these one of the more expensive BMPs to operate.

Many watersheds in Orange County are already highly developed and contribute to the impairment of the associated receiving water. Although the BMPs listed above should be considered first, pre-existing stormwater conveyance systems and the lack of available space in developed watersheds may greatly reduce the number and type of BMPs that may be appropriate for addressing identified impairments. Consequently, the following small footprint devices, may need to be considered, in addition to the conventional controls, for retrofit situations even though their pollutant removal performance may not be comparable to the best conventional public domain BMPs.

- Water Quality Inlets (enhanced catch basins)
- Proprietary End-of-Pipe Controls
- Proprietary Drain Inlet Inserts

The following BMP is not currently recommended for widespread implementation at this time.

- Retention / Irrigation

This technology is relatively new and has been implemented only in the Austin, Texas area. There is no published data on the operation, performance, and maintenance of this technology. In addition, the use of mechanical pumping and irrigation distribution systems may lead to higher maintenance requirements than many other devices.

### E1-5.6 BMP Selection Process

Although one would prefer base BMP selection primarily on the ability to reduce the concentrations and loads of constituents of concern for the local receiving water, this is not always possible. In many cases, the physical characteristics of a site drive the selection process. Important characteristics include the available hydraulic head, whether there is a perennial source of water, and the nature of the soil and subsurface conditions.

A BMP selection matrix was developed to guide the County and others in the selection of the most appropriate BMP for a given site. This matrix uses attributes such as tributary area, required head, soils, and other factors to narrow the final decision making to just a few select BMPs. This matrix is presented in Table E1-50. The optimum BMPs for the Orange County are shown under the heading “Preferred BMPs.” Where site specific conditions make implementation of these infeasible, BMPs listed under the heading “Secondary BMPs” might be considered.

### E1-5.6.1 Physical suitability

#### *E1-5.6.1.1 Catchment Area*

An important criterion in BMP selection is the size of the contributing watershed. Because of design constraints, such as outlet design, most BMPs are more suitable for either onsite or regional treatment. Consequently, once the catchment area is known many potential BMPs will be eliminated.

#### *E1-5.6.1.2 Soil Type*

As mentioned previously, the type of soil and the elevation of the groundwater table are important factors when considering implementation of infiltration devices. Hydrologic Soil Group A and B soils and a water table at least 10 feet below the proposed basin invert are absolutely necessary to avoid potential catastrophic failure of the infiltration ability of these BMPs. An *in-situ* soil permeability of 0.5 in / hr is recommended as a minimum when siting an infiltration device.

#### *E1-5.6.1.3 Hydraulic Head*

The amount of the available hydraulic head is an especially important factor in determining whether sand filters can be successfully installed. These devices often require 3- 4 feet of head for the sedimentation basin, filter bed, and underdrain. Retrofit situations often have very limited head which limits the types of BMPs that are appropriate.

#### *E1-5.6.1.4 Perennial Flow*

The availability of a perennial water supply to maintain a permanent pool in wetlands and constructed wetlands is normally a fundamental requirement. Although constructed wetlands in Laguna Niguel operate successfully with minimal water input, there are a couple of issues that may limit their use elsewhere, vector considerations and endangered species. Maintaining a permanent pool in wet ponds is necessary for the survival of mosquito fish introduced to control mosquito larvae. In addition, ephemeral pools are habitat for the fairy shrimp and their presence may greatly complicate future maintenance efforts.

### E1-5.6.2 Effluent Concentration

Where the physical characteristics of a site are appropriate for implementation of several different types of BMPs, achieving a low effluent concentration of the constituents of concern for the local receiving water should be the primary criterion for device selection. Chapter 4 compares the performance of various BMPs for a number of constituents and should be referred to for assessing the relative benefits of the BMPs under consideration. For many constituents, such as nutrients and bacteria the differences in effluent quality achieved by the various BMPs are not statistically significant.

In this matrix the effluent concentration has been characterized as “Low,” “Moderate,” and “High.” With respect to sediment, these terms would correspond to effluent concentrations of less than 20 mg/L, 20 – 50 mg/L, and greater than 50 mg/L on average.

### E1-5.6.3 Aesthetic Considerations

Aesthetics can be an important factor when the BMP will be clearly visible. Vegetated controls are not normally recognized as water quality devices by most people and, consequently, can be less objectionable than some other devices. Wet ponds are clearly one of the most popular choices from an aesthetic standpoint, since many find the open water and associated wildlife attractive as well as educational. On the other hand, sand filters and detention basins often lack aesthetic appeal.

### E1-5.6.4 Operation and Maintenance Requirements

Operation and maintenance requirements are reflected in the ongoing cost of operating stormwater BMPs; consequently, those with lower maintenance requirements are preferred, other factors being similar. For most BMPs the primary maintenance activities are those associated with vegetation maintenance, not with activities directly connected to BMP performance, such as sediment removal. Although vegetation management is important for the performance of vegetated controls, these grassy areas are comparable to landscaped areas that would receive routine mowing or weeding, so there may not be any real additional maintenance required than the area would receive on a normal basis.

Wet ponds are a special case for vegetation management, since the vector control agencies have required annual harvesting of plants at least at one site in the Orange County area. Harvesting is also a method by which nutrients are removed from the system. This harvesting can be quite time consuming and expensive, so care should be exercised when considering implementing this BMP.

E1-5.6.5 Area Required

The area required for the BMP footprint can be a factor in selection, if multiple choices are still available after other variables have been considered. The larger space requirements of some BMPs may limit the siting opportunities and increase their cost. Vegetated controls, such as buffer strips and bioretention systems, have some of the higher land requirements, but can often be incorporated into the site landscaping.

E1-5.6.6 References

Herricks, E., 1995. *Stormwater Runoff and Receiving Systems: Impact, Monitoring, and Assessment*, CRC Press, Boca Raton, FL.

**Table E1-50 BMP Selection Matrix**

| BMP Type | Catchment Area, ac | Soils | Required Head, ft | Perennial Flow | Effluent Concentration | Aesthetic Considerations | Area Required (% of watershed) |
|---|---|---|---|---|---|---|---|
| **Preferred BMPs** | | | | | | | |
| Extended Detention | 5 - 100 | Any | < 1 | No | Moderate | Negative | 0.5 – 2.0 |
| Swales | <10 | Any | < 1 | No | Moderate | Positive | 5 |
| Buffer Strips | <1 | Any | < 1 | No | Moderate | Positive | 5 – 10 |
| Bioretention | 0.25 - 1 | Any | < 1 | No | Low | Positive | 5 – 10 |
| Sand Filters | 1 - 25 | Any | 3-4 | No | Low | Negative | 1 – 4 |
| Infiltration Basins | 5 -100 | Group A or B | < 1 | No | Low | Negative | 0.5 – 2 |
| Infiltration Trenches | < 5 | Group A or B | < 1 | No | Low | Neutral | 0.5 – 2 |
| Wet Ponds | 25 - 200 | Any | < 1 [a] | Required | Low | Positive | 1 – 3 |
| Constructed Wetlands | 10 - 50 | Any | < 1 [a] | Required | Low | Negative | 2 – 6 |
| **Secondary BMPs** | | | | | | | |
| Water Quality Inlets | <1 | Any | < 1 | No | High [b] | Neutral | 0 |
| Proprietary End-of-Pipe | <20 | Any | < 1 | No | High [b] | Neutral | 0 |
| Drain Inlet Inserts | <1 | Any | < 1 | No | High [b] | Neutral | 0 |

[a] If permanent pool not gravity drained.
[b] Substantial variability among the various manufacturers.

**Glossary**

- **Best Available Technology Economically Achievable (BAT)**
  Technology-based standard established by the Clean Water Act (CWA) as the most appropriate means available on a national basis for controlling the direct discharge of toxic and nonconventional pollutants to navigable waters. BAT effluent limitations guidelines, in general, represent the best existing performance of treatment technologies that are economically achievable within an industrial point source category or subcategory.

- **Best Conventional Pollutant Control Technology (BCT)**
  Technology-based standard for the discharge from existing industrial point sources of conventional pollutants including BOD, TSS, fecal coliform, pH, oil and grease. The BCT is established in light of a two-part "cost reasonableness" test which compares the cost for an industry to reduce its pollutant discharge with the cost to a POTW for similar levels of reduction of a pollutant loading. The second test examines the cost-effectiveness of additional industrial treatment beyond BPT. EPA must find limits which are reasonable under both tests before establishing them as BCT.

- **Best Management Practice (BMP)**
  Permit condition used in place of or in conjunction with effluent limitations to prevent or control the discharge of pollutants. May include schedule of activities, prohibition of practices, maintenance procedure, or other management practice. BMPs may include, but are not limited to, treatment requirements, operating procedures, or practices to control plant site runoff, spillage, leaks, sludge or waste disposal, or drainage from raw material storage.

- **Biochemical Oxygen Demand (BOD)**
  A measurement of the amount of oxygen utilized by the decomposition of organic material, over a specified time period (usually 5 days) in a wastewater sample; it is used as a measurement of the readily decomposable organic content of a wastewater.

- **Chemical Oxygen Demand (COD)**
  A measure of the oxygen-consuming capacity of inorganic and organic matter

present in wastewater. COD is expressed as the amount of oxygen consumed in mg/l. Results do not necessarily correlate to the biochemical oxygen demand (BOD) because the chemical oxidant may react with substances that bacteria do not stabilize.

---

- **Chronic**
  A stimulus that lingers or continues for a relatively long period of time, often one-tenth of the life span or more. Chronic should be considered a relative term depending on the life span of an organism. The measurement of a chronic effect can be reduced growth, reduced reproduction, etc., in addition to lethality.

---

- **Clean Water Act (CWA)**
  The Clean Water Act is an act passed by the U.S. Congress to control water pollution. It was formerly referred to as the Federal Water Pollution Control Act of 1972 or Federal Water Pollution Control Act Amendments of 1972 (Public Law 92-500), 33 United States Code 1251 et. seq., as amended by: Public Law 96-483; Public Law 97-117; Public Laws 95-217, 97-117, 97-440, and 100-04.

---

- **Code of Federal Regulations (CFR)**
  A codification of the final rules published daily in the Federal Register. Title 40 of the CFR contains the environmental regulations.

---

- **Composite Sample**
  Sample composed of two or more discrete samples. The aggregate sample will reflect the average water quality covering the compositing or sample period.

---

- **Conventional Pollutants**
  Pollutants typical of municipal sewage, and for which municipal secondary treatment plants are typically designed; defined by Federal Regulation [40 CFR 401.16] as BOD, TSS, fecal coliform bacteria, oil and grease, and pH.

---

- **Criteria**
  The numeric values and the narrative standards that represent contaminant concentrations that are not to be exceeded in the receiving environmental media (surface water, ground water, sediment) to protect beneficial uses.

- **Discharge Monitoring Report (DMR)**
  The form used (including any subsequent additions, revisions, or modifications) to report self-monitoring results by NPDES permittees. DMRs must be used by approved states as well as by EPA.

- **Grab Sample**
  A sample which is taken from a waste stream on a one-time basis without consideration of the flow rate of the waste stream and without consideration of time.

- **Municipal Separate Storm Sewer System (MS4)**
  A conveyance or system of conveyances (including roads with drainage systems, municipal streets, catch basins, curbs, gutters, ditches, manmade channels, or storm drains) owned by a state, city, town or other public body, that is designed or used for collecting or conveying stormwater, which is not a combined sewer, and which is not part of a publicly owned treatment works. Commonly referred to as an "MS4" [40 CFR 122.26(b)(8)].

- **Municipal Sources**
  POTWs collect domestic sewage from houses, other sanitary wastewater, and wastes from commercial and industrial facilities. POTWs discharge conventional pollutants, and are covered by secondary treatment standards and state water quality standards. POTWs also produce biosolids during the treatment process.

- **National Pollutant Discharge Elimination System (NPDES)**
  The national program for issuing, modifying, revoking and reissuing, terminating, monitoring and enforcing permits, and imposing and enforcing pretreatment requirements, under Sections 307, 318, 402, and 405 of CWA.

- **pH**
  A measure of the hydrogen ion concentration of water or wastewater; expressed as the negative log of the hydrogen ion concentration in mg/l. A pH of 7 is neutral. A pH less than 7 is acidic, and a pH greater than 7 is basic.

- **Point Source**
  Any discernible, confined, and discrete conveyance, including but not limited to any pipe, ditch, channel, tunnel, conduit, well, discrete fixture, container, rolling stock, concentrated animal feeding operation, landfill leachate collection system, vessel, or other floating craft from which pollutants are or may be discharged.

---

- **Pollutant**
  Dredged spoil, solid waste, incinerator residue, filter backwash, sewage, garbage, sewage sludge, munitions, chemical wastes, biological materials, radioactive materials (except those regulated under the Atomic Energy Act of 1954, as amended (42 United States Code 2011 et seq.)), heat, wrecked or discarded equipment, rock, sand, cellar dirt and industrial, municipal, and agricultural waste discharged into water.

---

- **Pretreatment**
  The reduction of the amount of pollutants, the elimination of pollutants, or the alteration of the nature of pollutant properties in wastewater prior to or in lieu of discharging or otherwise introducing such pollutants into a publicly owned treatment works [40 CFR 403.3(q)].

---

- **Primary Treatment**
  The practice of removing some portion of the suspended solids and organic matter in a wastewater through sedimentation. Common usage of this term also includes preliminary treatment to remove wastewater constituents that may cause maintenance or operational problems in the system (i.e., grit removal, screening for rags and debris, oil and grease removal, etc.).

---

- **Priority Pollutants**
  Those pollutants considered to be of principal importance for control under the CWA based on the NRDC consent decree settlement [( NRDC et al. v. Train, 8 E.R.C. 2120 (D.D.C. 1976), modified 12 E.R.C. 1833 (D.D.C. 1979)]; a list of these pollutants is provided as Appendix A to 40 CFR Part 423.

---

- **Proposed Permit**
  A state NPDES permit prepared after the close of the public comment period

(and when applicable, any public hearing and administrative appeals) which is sent to EPA for review before final issuance by the state.

- **Stormwater**
  Stormwater runoff, snow melt runoff, and surface runoff and drainage [40 CFR 122.26(b)(13)].

- **Total Maximum Daily Load (TMDL)**
  The amount of pollutant, or property of a pollutant, from point, nonpoint, and natural background sources, that may be discharged to a water quality-limited receiving water. Any pollutant loading above the TMDL results in violation of applicable water quality standards.

- **Total Suspended Solids (TSS)**
  A measure of the filterable solids present in a sample, as determined by the method specified in 40 CFR Part 136.

- **Toxic Pollutant**
  Pollutants or combinations of pollutants, including disease-causing agents, which after discharge and upon exposure, ingestion, inhalation or assimilation into any organism, either directly from the environment or indirectly by ingestion through food chains, will, on the basis of information available to the Administrator of EPA, cause death, disease, behavioral abnormalities, cancer, genetic mutations, physiological malfunctions, (including malfunctions in reproduction) or physical deformations, in such organisms or their offspring. Toxic pollutants also include those pollutants listed by the Administrator under CWA Section 307(a)(1) or any pollutant listed under Section 405(d) which relates to sludge management.

- **Toxicity Test**
  A procedure to determine the toxicity of a chemical or an effluent using living organisms. A toxicity test measures the degree of effect on exposed test organisms of a specific chemical or effluent.

- **Water Quality Criteria**
  Comprised of numeric and narrative criteria. Numeric criteria are scientifically

derived ambient concentrations developed by EPA or states for various pollutants of concern to protect human health and aquatic life. Narrative criteria are statements that describe the desired water quality goal.

---

- **Water Quality Standard (WQS)**
  A law or regulation that consists of the beneficial use or uses of a waterbody, the numeric and narrative water quality criteria that are necessary to protect the use or uses of that particular waterbody, and an antidegradation statement.

---

- **Waters of the United States**
  All waters that are currently used, were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters subject to the ebb and flow of the tide. Waters of the United States include but are not limited to all interstate waters and intrastate lakes, rivers, streams (including intermittent streams), mudflats, sand flats, wetlands, sloughs, prairie potholes, wet meadows, play lakes, or natural ponds. [See 40 CFR 122.2 for the complete definition.]

---

# *ATTACHMENT A*

## *NPDES Permit Requirements for Structural Controls*

**B. WATER QUALITY MANAGEMENT PLAN (WQMP) FOR URBAN RUNOFF (FOR NEW DEVELOPMENT/SIGNIFICANT REDEVELOPMENT):**

By March 1, 2003, the permittees shall review their existing BMPs for New Developments (Appendix G of the DAMP) and submit for review and approval by the Executive Officer, a revised WQMP for urban runoff from new developments / significant re-developments for the type of projects listed below:

a. All significant re-development projects, where significant re-development is defined as the addition of 5,000 or more square feet of impervious surface on an already developed site. This includes additional buildings and / or structures, extension of existing footprint of a building, construction of parking lots, etc.

b. Home subdivisions of 10 units or more. This includes single family residences, multi-family residences, condominiums, apartments, etc.

c. Commercial and industrial developments of 100,000 square feet or more. This includes non-residential developments such as hospitals, educational institutions (to the extent the permittees have authority to regulate these developments), recreational facilities, mini-malls, hotels, office buildings, warehouses, and light & heavy industrial facilities.

d. Automotive repair shops (with SIC codes 5013, 5014, 5541, 7532-7534, 7536-7539).

e. Restaurants where the land area of development is 5,000 square feet or more.

f. All hillside developments on 10,000 square feet or more, which are located on areas with known erosive soil conditions or where the natural slope is twenty-five percent or more.

g. Developments of 2,500 square feet of impervious surface or more adjacent to (within 200 feet) or discharging directly into environmentally sensitive areas, such as areas designated in the Ocean Plan as areas of special biological significance or waterbodies listed on the CWA Section 303(d) list of impaired waters.

h. Parking lots of 5,000 square feet or more exposed to stormwater. Parking lot is defined as a land area or facility for the temporary storage of motor vehicles.

7. The permittees are encouraged to include in the WQMP the development and implementation of regional and / or watershed management programs that address runoff from new development and significant re-development. The WQMP shall include BMPs for source control, pollution prevention, and / or structural treatment BMPs. For all structural treatment controls, the WQMP shall identify the responsible party for maintenance of the treatment system, and a funding source or sources for its operation and maintenance. The goal of the WQMP is to develop and implement practicable programs and policies to minimize the effects of urbanization on site hydrology, urban runoff flow rates or velocities and pollutant loads. This goal may be achieved through watershed-based structural treatment controls, in combination with site-specific BMPs. The WQMP shall reflect consideration of the following goals, which may be addressed through on-site-and / or watershed-based BMPs.

    a. The pollutants in post-development runoff shall be reduced using controls that utilize best available technology (BAT) and best conventional technology (BCT).

    b. The discharge of any listed pollutant to an impaired waterbody on the 303(d) list shall not cause an exceedance of receiving water quality objectives.

8. During the time that the WQMP is being revised, the permittees shall implement their existing requirements for new development (Appendix G of the DAMP). If the Executive Officer does not approve the revised WQMP by October 1, 2003, as meeting the goals proposed in XII.B.2, above and providing an equivalent or superior degree of treatment as the sized criteria outlined in XII.B.3.A, B and C, below, structural BMPs shall be required for all new development and significant redevelopment[1]. Minimum structural BMPs must either be sized to comply with one of the following numeric sizing criteria or be deemed by the Principal Permittee to provide equivalent or superior treatment, either on a site basis or a watershed basis:

A. Volume

[1] Where new development is defined as projects for which tentative tract or parcel map approval was not received by July 1, 2003 and new re-development is defined as projects for which all necessary permits were not issued by July 1, 2003. New development does not include projects receiving map approvals after July 1, 2003 that are proceeding under a common scheme of development that was the subject of a tentative tract or parcel map approval that occurred prior to July 1, 2003.

Volume–based BMPs shall be designed to infiltrate, filter, or treat either:

1. The volume of runoff produced from a 24-hour, 85th percentile storm event, as determined from the local historical rainfall record; or,

2. The volume of annual runoff produced by the 85th percentile, 24-hour rainfall event, determined as the maximized capture stormwater volume for the area, from the formula recommended in Urban Runoff Quality Management, WEF Manual of Practice No. 23/ ASCE Manual of Practice No. 87 (1998); or,

3. The volume of annual runoff based on unit basin storage volume, to achieve 80% or more volume treatment by the method recommended in California Stormwater Best Management Practices Handbook – Industrial / Commercial (1993); or,

4. The volume of runoff, as determined from the local historical rainfall record, that achieves approximately the same reduction in pollutant loads and flows as achieved by mitigation of the 85th percentile, 24-hour runoff event;

OR

B. Flow

Flow-based BMPs shall be designed to infiltrate, filter, or treat either:

7. The maximum flow rate of runoff produced from a rainfall intensity of 0.2 inch of rainfall per hour; or,

8. The maximum flow rate of runoff produced by the 85th percentile hourly rainfall intensity, as determined from the local historical rainfall record, multiplied by a factor of two; or,

9. The maximum flow rate of runoff, as determined from the local historical rainfall record that achieves approximately the same reduction in pollutant loads and flows as achieved by mitigation of the 85th percentile hourly rainfall intensity multiplied by a factor of two.

C. Groundwater Protection

Any structural infiltration BMPs shall meet the following minimum requirements:

1. Use of structural infiltration treatment BMPs shall not cause or contribute to an exceedance of groundwater water quality objectives.

2. Source control and pollution prevention control BMPs shall be implemented to protect groundwater quality.

3. Structural infiltration treatment BMPs shall not cause a nuisance or pollution, as defined in Water Code Section 13050.

4. The permittees may propose any equivalent sizing criteria for treatment BMPs or other controls that will achieve greater or substantially similar pollution control benefits. In the absence of approved equivalent sizing criteria, the permittees shall implement the above stated sizing criteria.

5. If a particular BMP is not technically feasible, other BMPs should be implemented to achieve the same level of compliance, or if the cost of BMP implementation greatly outweighs the pollution control benefits, the permittees may grant a waiver of the numeric sizing criteria. All waivers, along with waiver justification documentation, must be reported to the Regional Board in writing within 30 days. The permittees may propose to establish an urban runoff fund to be used for urban water quality improvement projects within the same watershed that is funded by contributions from developers granted waivers. If it is determined by the Regional Board that waivers are being inappropriately granted, this Order may be reopened to modify these waiver conditions.

6. The obligation to install minimum structural BMPs at new development is met if, for a common scheme of development, BMPs are constructed with the requisite capacity to serve the entire common scheme, even if certain phases of the common scheme may not have BMP capacity located on that phase in accordance with the requirements specified above.

# *ATTACHMENT B*

## *Volume Reducing BMPs for Short-Term Groundwater-Related Discharges*

**Volume Reducing Best Management Practices**
**For Short-Term Groundwater-Related Discharges Within Orange County**
**(Santa Ana Region)**

## Background

In 1998, the Santa Ana Regional Water Quality Control Board (Regional Board) adopted a general National Pollutant Discharge Elimination System (NPDES) permit (Order No. 98-67) prescribing general waste discharge requirements for discharges to surface waters that pose an insignificant (*de minimus*) threat to water quality within the Santa Ana Region.

In mid-2003, Order No. 98-67 was renewed by Order No. R8-2003-0061 (Region Wide General De Minimus Permit). Although the terms and conditions were relatively the same, Order No. R8-2003-0061 specifically excluded groundwater-related discharges within the San Diego Creek/Newport Bay watershed (refer to **Figure 1**). Instead, the Order found that these discharges would continue to be covered under Order No. 98-67 until separate waste discharge requirements were adopted. This regulatory approach was taken in light of the concern that the groundwater-related discharges in the San Diego Creek/Newport Bay watershed have the potential to adversely affect surface waters within the watershed and may not comply with established nutrient and toxic Total Maximum Daily Loads (primarily due to nitrate and selenium concentrations).

Therefore, in order to regulate discharges in the San Diego Creek/Newport Bay watershed separately, the Regional Board adopted Order No. R8-2004-0021 on December 20, 2004 (San Diego Creek/Newport Bay De Minimus Permit). This Order specifies waste discharge requirements for short-term groundwater-related discharges and for *de minimus* discharges within the San Diego Creek/Newport Bay watershed. The Order specifically regulates the following types of discharges:

1. Short term (one year or less duration) discharges from activities involving groundwater extraction and discharge:
    a. Wastes associated with well installation, development, test pumping and purging;
    b. Aquifer testing wastes;
    c. Groundwater-related wastes from subterranean seepage; and
    d. Groundwater-related wastes at construction sites.

2. Discharges that pose an insignificant threat to water quality:
    a. Construction dewatering wastes not involving groundwater (except storm water dewatering at construction sites);
    b. Discharges resulting from hydrostatic testing of vessels, pipelines, tanks, etc.;
    c. Discharges resulting from the maintenance of potable water supply pipelines, tanks, reservoirs, etc.;
    d. Discharges resulting from the disinfection of potable water supply pipelines, tanks, reservoirs, etc.;
    e. Discharges from potable water supply systems resulting from system failures, pressure releases, etc.;
    f. Discharges from fire hydrant testing or flushing;
    g. Non-contact cooling water;
    h. Air conditioning condensate;
    i. Swimming pool drainage;
    j. Discharges resulting from diverted stream flows;
    k. Discharges from residential sump pumps; and
    l. Other similar types of wastes, which pose a de minimus threat to water quality, yet technically must be regulated under waste discharge requirements.

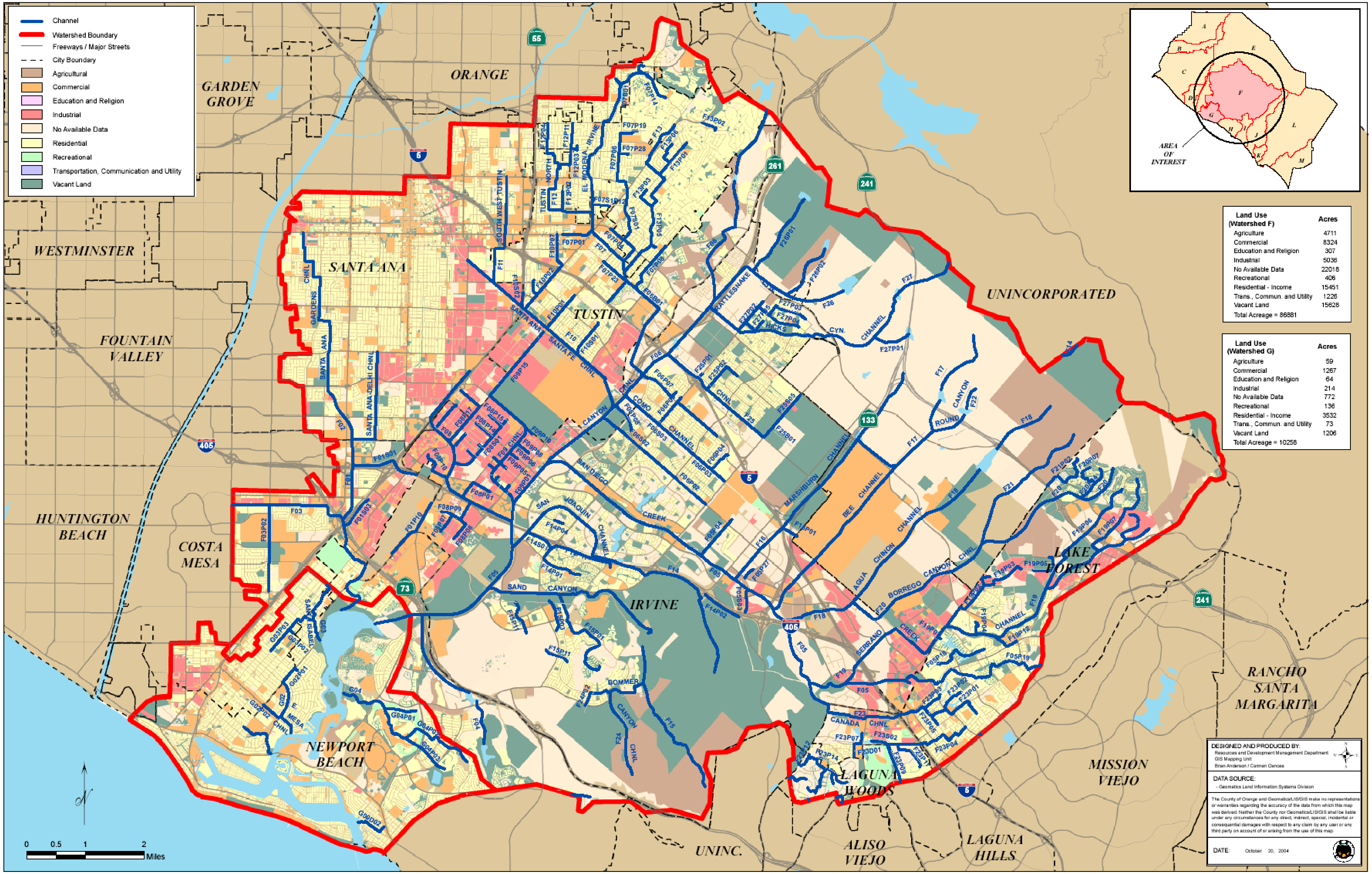



**Figure 1. San Diego Creek/Newport Bay Watershed Map**

(for more information refer to the Orange County Watersheds website at www.ocwatersheds.com).

Order No. R8-2004-0021 also recognizes that:

a. For nitrates, available data indicates that the Irvine Ranch Water District's operation of the San Joaquin Marsh pond treatment system offsets discharges of nitrogen resulting from the groundwater-related discharges; and
b. For selenium, the dischargers cannot be assured, at the present time, of achieving numeric effluent limitations through reasonable treatment, source control, or pollution prevention measures as such measures are not currently available for short-term groundwater discharges containing selenium. Moreover, it may not be practical to implement such technology, if and when it becomes available, for the short-term groundwater discharges. **Figures 2** and **3** below show the estimated region of shallow groundwater in the San Diego Creek Watershed and selenium concentrations in this area, respectively.


**Figure 2**. Map of estimated region of shallow groundwater in the San Diego Creek watershed.

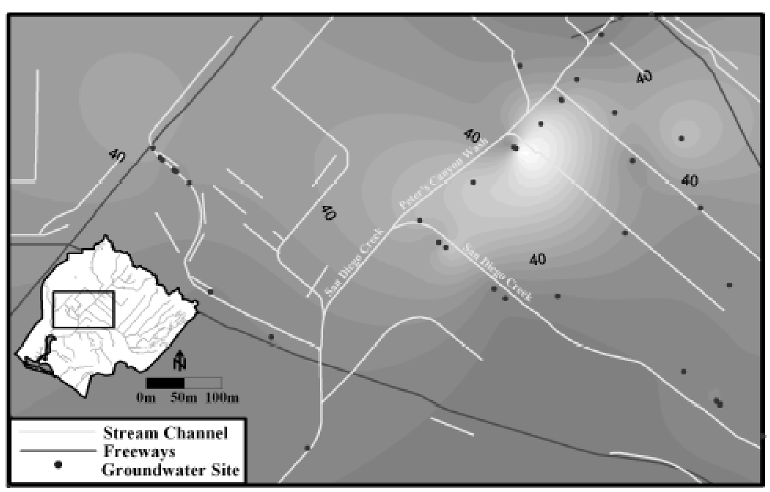



**Figure 3**. Concentration of selenium in groundwater near Peter's Canyon and Como Channels.

However, the Order also recognizes that the dischargers may be able to reduce or even eliminate nitrate and selenium discharges by implementing short-term volume-reduction measures and requires the dischargers to investigate potential BMPs as a part of the application process for coverage under the De Minimus Permit.

## De Minimus Permit Working Group and Work Plan

Given the uncertainties with regard to the availability and efficacy of treatment technologies, the need to investigate those technologies and discharge volume reduction measures, and the lack of available data necessary to identify baseline loadings resulting from groundwater inflows in the watershed, etc., certain dischargers subject to Order No. R8-2004-0021 (refer to **Table 1** below) have agreed to form a Working Group and have committed to fund and participate in a Work Plan. The purpose of the Work Plan is, in part, to develop a comprehensive understanding of and management plan for nitrate and selenium discharges to surface waters within the Newport Bay watershed that result from groundwater-related inflows.

**Table 1. De Minimus Working Group Participants (as of November 16, 2004).**

| Participant Name |
|---|
| California Department of Transportation |
| County of Orange |
| Orange County Flood Control District |
| City of Costa Mesa |
| City of Irvine |
| City of Laguna Hills |
| City of Laguna Woods |
| City of Lake Forest |
| City of Newport Beach |
| City of Orange |
| City of Santa Ana |
| City of Tustin |
| Irvine Ranch Water District |
| The Irvine Company |
| Southern California Water Company |
| Tustin Legacy Community Partners |

The De Minimus Permit includes different regulatory approaches for dischargers depending on whether or not a discharger is participating in the De Minimus Permit Working Group. For dischargers who demonstrate that compliance with the numeric selenium effluent limitations is infeasible and who *do* participate in the Working Group, the Permit allows the groundwater-related discharge provided that the De Minimus Working Group implements the Work Plan in a timely manner and that the discharger fulfills its financial and participatory requirements established by the Working Group. For dischargers who are *not* participating in the Working Group and cannot comply with the numeric effluent limitations, the discharger must either not proceed with the planned discharge or must identify and participate in a program that assures that selenium discharges in excess of those allowed by the Permit are offset on at least a one-to-one basis; the discharger would be required to identify a plan and schedule for implementation of the offset prior to starting any new discharge.

The volume reducing BMPs discussed in the following section will help some dischargers not only to determine which BMPs may be feasible for their projects, but also whether or not it may be beneficial to join the De Minimus Permit Working Group.

**Overview of Volume Reducing BMPs**

One of the first tasks in the Work Plan is to develop fact sheets for volume reduction BMPs. The De Minimus Permit requires dischargers to either implement appropriate volume reducing BMPs or to prove that the discharge cannot be reduced or eliminated through reasonable volume-reduction measures. These evaluations, including both the potential positive and negative impacts of the BMPs, must be investigated in an aggressive manner by potential dischargers. This requirement applies to all potential dischargers, including those who are also members of the Working Group.

Therefore, the purpose of this task is to identify and evaluate reasonable BMPs that may be used for reducing or possibly eliminating the volume of groundwater (and any associated selenium or nitrates contained in the groundwater) discharged to surface waters. This analysis will assist dischargers, both public agencies and private contractors, in evaluating compliance options for each individual project in the San Diego Creek/Newport Bay watershed.

It is important to note that the focus of this BMP evaluation is on volume-reduction measures related to short-term groundwater-related discharges. However, these BMPs may be applicable to other types of de minimus discharges regulated under the De Minimus Permit.

The fact sheets (presented in **ATTACHMENT A**), summarize the applicability, constraints, limitations and estimated costs for the following BMPs:

a. Discharges to land;
b. Evaporation ponds;
c. Discharges to the sanitary sewer;
d. Offsite transportation; and
e. Re-injection.

**Table 2** below can be used by dischargers to help determine which volume-reduction BMPs may be most appropriate for a specific project. Note that when considering the different volume reducing BMPs, discharge to the sanitary sewer should be considered only as a last resort. Even then, the sanitary sewer agencies are under no obligation to accept the discharge and may choose not accept such dewatering discharges.

**Table 2. Comparison of Volume-Reduction BMPs**

| Consideration | Discharge to Land | Discharge to Sewer | Evaporation Pond | Off Site Transport and Disposal | Reinjection |
|---|---|---|---|---|---|
| GENERAL APPLICABILITY | Sites with available land | Sites where other options not feasible and sewer agency will accept the discharge | Sites with available land | Sites with smaller discharges | Sites with smaller discharges & suitable soil conditions |
| CAPACITY | 1 gal/day + (depending on land available) | 8,000 gal/day to 1,000,000 gal/day | 1,000 gal/day+ (depending on footprint available) | 2,500 gal/day to 8,000 gal/day | 7,200 gal/day+ (depending on number of wells) |
| COST PER MILLION GALLONS | $17,000 – $26,000 | $8,000 – $75,000 | $50,000 – $78,000 | $129,000 – $135,000 | $220,000 – $1.2M (depending on number of wells) |
| REQUIRES OPEN LAND | ✓ | | ✓ | | |
| REQUIRES PERMIT | ✓ (RWQCB permit or waiver) | ✓ (sewer agency permit) | ✓ (RWQCB permit or waiver) | | ✓ (RWQCB permit if it occurs outside of project site) |
| REQUIRES MONITORING | ✓ (if land discharge permit required) | ✓ | ✓ (if unlined and land discharge permit required) | ✓ | ✓ (if it occurs outside of project site) |
| REQUIRES VECTOR MONITORING | | | ✓* | | |

* If water remains in pond for greater than 72 hours contact Vector Control Agency to determine requirements.
(see references at end of each fact sheet for sources of capacity and cost information)

In order to assist potential dischargers, a decision tree was developed (**ATTACHMENT B)** to help determine whether or not a planned discharge is subject to regulation under either of the two Santa Ana Regional Board's De Minimus General Permits, identify the appropriate permit that will regulate the discharge, and identify a few key provisions necessary for permit compliance.

# ATTACHMENT A
# Volume Reducing BMP Fact Sheets

# Discharge to Land

## Considerations

- Depending on land availability and suitability, can accommodate large volumes of water
- Cost \$0.017/gal to \$0.026/gal (greater if testing is required)
- Project site should be large enough to accommodate the discharge to land
- May require discharge to land permit
- Monitoring may be required if discharge to land permit is required
- Water should not leave the site where it is spread

## Description

A discharge to land typically involves collecting the water from the site, potentially transporting it and then spraying or spreading water onto the same or another project site, where it is infiltrated, evaporated, or taken in by plants. Water may also be used for dust control or irrigation.

Water wastes discharged to land may be required to comply with the Statewide General Waste Discharge Requirements for Discharges to Land with a Low Threat to Water Quality (Order No. 2003-0003-DWQ). This "Land Discharge General Permit" is a statewide permit that specifies restrictions for various types of discharges to land. For certain types of discharges within the Santa Ana Region (see Table 3 below), dischargers may be able to obtain a waiver from the need to obtain coverage under the Land Discharge General permit, but only after submitting a written request for consideration by Regional Board staff.

**Table 3 Discharges Regulated by Statewide Land Discharge General Permit**

| Discharge Type | Santa Ana Regional Board Waiver Possible |
|---|---|
| **Wells/Boring Waste** | |
| Well Development Discharge | |
| Monitoring Well Purge Water Discharge | X |
| Boring Waste Discharge | X |
| **Clear Water Discharges** | |
| Water Main/Water Storage Tank / Water Hydrant Flushing | X |
| Pipelines/Tank Hydrostatic Testing Discharge | X |
| Commercial and Public Swimming Pools | |
| **Small Dewatering Projects** | |
| Small/Temporary Dewatering Projects (such as excavations during construction) | X |
| **Miscellaneous** | |
| Small Inert Solid Waste Disposal Operations | X |
| Cooling Discharge | X |

# Discharge to Land

### Advantages

- Coverage under the San Diego Creek/Newport Bay De Minimus Permit or Region Wide General De Minimus Permit is not required as there is no discharge to surface waters.

- Reuse of discharges helps conserve water if used for irrigation or dust control.

### Limitations

- Spreading site must be amenable to infiltration (e.g., of sufficient grade, soil type, soil condition, and/or vegetation level) so that spread water will infiltrate, evaporate, or be taken in by plant materials. For example, sites with substantial grade, clay soils, saturated soils, insufficient space and/or insufficient vegetation may not be amenable to spreading.

- May require coverage under Land Discharge General Permit or a waiver from the Santa Ana Regional Board (a written request for the discharge or waiver must be filed for consideration).

- Monitoring for Title 22 constituents may be required if there are known contaminants. If levels are higher than those shown in Table 4 then discharge to land is prohibited.

- An annual fee will be required if coverage under the Land Discharge General Permit is required. The annual fee for most short-term operations ranges from $800 to $3,400.

**Table 4 Title 22 Constituent Concentrations to be Considered Hazardous**

| Constituent | Maximum for any single composite sample (mg/L) |
|---|---|
| Antimony | 0.1 |
| Arsenic | 0.5 |
| Barium | 7.6 |
| Beryllium | 0.01 |
| Cadmium | 0.05 |
| Chromium (total) | 0.33 |
| Lead | 0.15 |
| Mercury | 0.009 |
| Nickel | 1 |
| Selenium | 0.16 |
| Silver | 0.3 |
| Thallium | 0.02 |
| Zinc | 70 |

- Onsite collection and storage equipment may be required to hold water prior to spreading.

- Due to the noise generated by the pumps, there may be restrictions on hours of operation.

### Requirements

- Dischargers must seek coverage under the Land Discharge General Permit or seek a waiver from the Santa Ana Regional Board by filing:

(a) A Notice of Intent (NOI) to comply with the terms and conditions of the General Waste Discharge Requirements (WDRs) or a Report of Waste Discharge (ROWD);

(b) The applicable first annual fee as required by Title 23, California Code of Regulations (CCR), Section 2200;

(c) A project map;

(d) Evidence of California Environmental Quality Act (CEQA) compliance; and

(e) A discharger monitoring plan.

The Discharge monitoring plans should include a list of all pollutants believed to be present in the discharge, approximate concentration of pollutants, monitoring location and frequencies and a report schedule. The Regional Board will have different requirements based on the size of the project and volume of water.

Upon review by Regional Board staff, a determination will be made as to whether or not coverage under the General WDRs is appropriate. The discharger will be notified by a letter from the Regional Board Executive Officer when coverage under the General WDRs has been approved.

- Discharge of any waste to surface waters is prohibited.
- Discharge to lands not owned or controlled by the dischargers is prohibited, unless the discharger has a written agreement with the property owner.
- Discharges of waste classified as "hazardous" or "designated" as defined in Title 22 CCR, Section 66261 and CWC Section 13173, are prohibited (refer to Table 4 for Title 22 concentrations).
- Pumping equipment (and possibly storage equipment) is required to transfer discharge water to the water truck (and store prior to discharge to land as necessary). Site work (e.g. discing or berming) may also be needed.
- Water truck and operator are needed to discharge water by spraying.

### Cost Considerations

Cost considerations associated with this BMP include pumping equipment, water truck (storage containers as necessary) and operator. Discharge to land is generally the least expensive volume reduction practice of the four discussed, provided there is sufficient land within the project site to discharge water. For more detailed cost information, contact a company that specializes in pumps and portable sprinkler lines. Most pumps are powered by diesel engines that will require daily maintenance checks and fuel. Table 5 below provides an example of typical expenses to expect when utilizing the Discharge to Land BMP for a project with a total anticipated discharge of 1,000,000 gallons.

# Discharge to Land

**Table 5 Discharge to Land Cost Example**

| Item | Cost | Gallons to be Disposed | Cost / Gallon |
|---|---|---|---|
| Pumping | $50/hr | 9,000 / hr | $0.005 / gal |
| Water Quality Testing | $600 | Prior to initial discharge and as needed | - |
| Permit Fee | $3,400 | Annual Fee | - |
| Operator and Water Truck | $150 / hr | 9,000 /hr | $0.017/gal |
| | Total Cost | for 1,000,000 gallons | $26,200 |

### References and Sources of Additional Information

Department of Transportation; October 2001, *Field Guide to Construction Site Dewatering* CTSW-RT-01-010

State Water Resources Control Board; 2003, *Water Quality Order No. 2003-0003-DWQ* (http://www.waterboards.ca.gov/resdec/wqorders/2003/wqo/wqo2003-0003.pdf)

# Discharge to Sanitary Sewer

### Considerations

- Volume of water (8,000 to 1,000,000 gal/day)
- Cost $0.008/gal to $0.075/gal
- Requires a permit from local sanitary sewer agency
- Local sanitary sewer must be willing to accept discharge—of acceptable quantity and quality

### Description

This practice consists of discharging water directly to the sanitary sewer system instead of surface waters or the storm drain system. Discharge to the sanitary sewer requires permission from the appropriate sanitary sewer agency, and the agency may require a permit and/or a fee. The permit is granted to allow the discharger to discharge the sewage facilities (via sanitary sewer or at wastewater treatment facility). The discharger must take appropriate measures to protect the sewerage facility. Sanitary sewer agencies within Orange County are shown in Figure 4. A San Diego Creek Watershed Jurisidictional Map for IRWD service areas is shown in Figure 5.

### Advantages

- Coverage under the San Diego Creek/Newport Bay De Minimus Permit or Region Wide General De Minimus Permit is not required as there is no discharge to surface waters.
- The sewer agency will determine acceptable levels of pollutants.
- At certain locations water may be pumped directly to the sanitary sewer. Other locations may require transportation to a sanitary sewer connection.

### Limitations

- Discharge may require treatment for sediment removal or removal/reduction of other pollutants (e.g. heavy metals, hydrocarbons etc.) as required by the sewer agency. Any necessary treatment will increase costs of disposal (possibly substantially).
- Requires approval from the local sewer agency. A fee and/or permit will be required from the local sewer agency.
- Discharge to the sanitary sewer may not be allowed during wet weather periods.
- Water quality testing and flow estimation are typically required prior to discharge to sanitary sewer, and throughout the duration of the permit to discharge to the sanitary sewer.

# Discharge to Sanitary Sewer

### Requirements

- An agreement or permission (possibly in the form of a permit) from the sanitary sewer agency is required. Figure 4 shows the jurisdictions of the major sanitary sewer agencies within Orange County.

    1. Orange County Sanitation District (OCSD) requires a Special Purpose Discharge Permit. The discharge permits may be granted when no alternative method of disposal is reasonably available, or to mitigate an environmental risk or health hazard. It usually takes 30-60 days to receive a permit after the application has been filed. (Refer to Ordinance No. 1 located at www.ocsd.com/about/ordinances.asp). Levels of pollutants in the discharge must be acceptable to OCSD.

    2. Irvine Ranch Water District (IRWD) and OCSD issue joint Special Purpose Discharge Permits. The discharge permits may be granted when no alternative method of disposal is reasonably available. The applicant bears the burden of demonstrating that no alternative method is reasonably available (www.irwd.com/WaterService/Rules& &Regulations2004.pdf). Levels of pollutants in the discharge must be acceptable to IRWD and its water recycling program. It usually takes 30-60 days to receive a permit after an application is determined to be complete.

    3. South Orange County Wastewater Authority (SOCWA) requires a Special Wastewater Discharge Permit and a letter of request from the Regional Board for any groundwater discharge into their member agency facilities. SOCWA has a cap of 5000 gal/day, and levels of pollutants in the discharge must be acceptable to SOCWA.

- To obtain an agreement with the sanitary sewer agency it may be necessary to provide the following information:

    - Water quality (for both OCSD and IRWD refer to http://www.ocsd.com/about/ordinances.asp. IRWD requirements cross-reference the requirements listed in the OCSD ordinance.)

    - Estimated flow

    - Duration of flow

- Monitoring of discharge and reporting of results is required by IRWD and OCSD as part of their agreements. Water quality testing of discharge should be performed in accordance with sanitary sewer agency policy and results provided prior to discharge to sanitary sewer. (OCSD and IRWD require monitoring for all constituents contained in the most current EPA "Priority Pollutant" list, excluding asbestos)

- The sanitary sewer agency will require records of discharges of water quality and quantity.

- Water may need to be treated for sediment (or pretreated for other pollutants, such as nitrates) prior to discharge to the sanitary sewer. If sediment filtration is required, the sediment must be properly managed, either dispersed on-site or properly disposed.

# Discharge to Sanitary Sewer

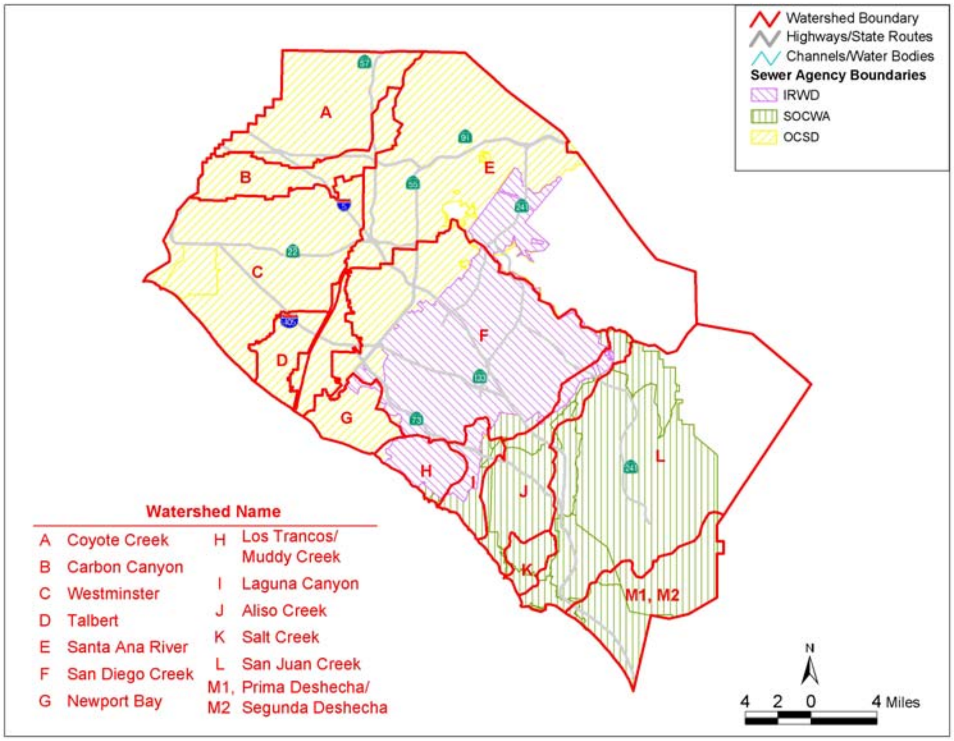



**Figure 4. Sewer Agency Jurisdictions within Orange County**

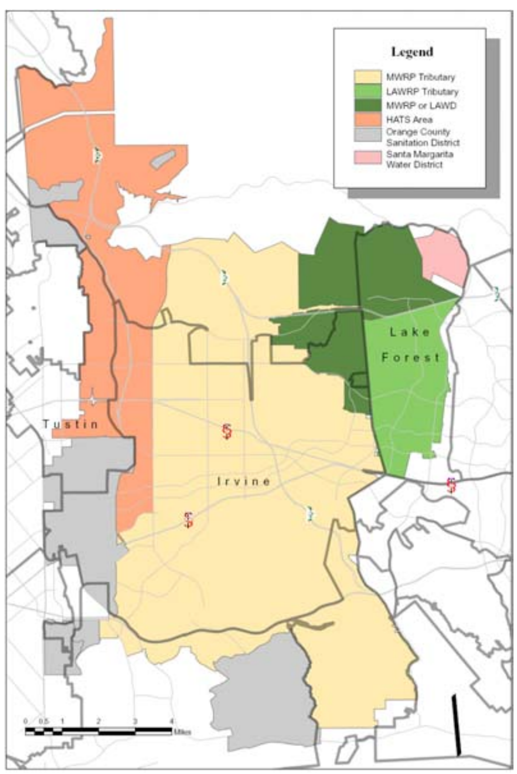



Figure 5. San Diego Creek Watershed Jurisdictional Map Within IRWD Service Area

# Discharge to Sanitary Sewer

## Cost Considerations

In most cases, local sanitary sewer agencies require a fee to discharge to the sewer (OCSD is $750 for permit and $637.55/million gallons discharged; IRWD permit fee is determined on a case-by-case basis and is similar to Regional Board costs for similar permits; SOCWA fee is $1,000/month; contact the appropriate sewer agency as fees may change over time). Additional costs include cost to pump water, water quality testing and sediment removal (and pretreatment for other pollutants as necessary). Costs of sediment removal, and any pretreatment necessary have not been included in the table below; these costs can be substantial and should be considered prior to any decision to discharge to the sanitary sewer. There may also be a fee to use the sanitary sewer system as well as a Capital Facility Recovery Fee (CORF) if the discharge is for a period longer than a few months. Discharge to the sewer can be a relatively expensive practice for disposal of groundwater-related discharges, particularly if water will need to be transported to a disposal site.

**Table 6** provides an example of typical costs to be expected for a project in OCSD jurisdiction that requires discharging 1,000,000 gallons to the sanitary sewer.

**Table 6 Cost Example**

| Item | Cost | Gallons to be Disposed | Cost / Gallon |
|---|---|---|---|
| Permit fees | $750 | One time fee | - |
| Water Quality Testing | $600 | Prior to discharge | - |
| Additional per gallon fee | $637.55 | Per 1,000,000 gallons | - |
| Pumping | $50/hr | 9,000 gallons / hr | $0.0055 / gal |
| Transport fees (if necessary) | $200 | Per 3,000 gallons | $0.067 / gal |
| | Total Cost | for 1,000,000 gallons | $74,500 |

## References and Sources of Additional Information


California Stormwater Quality Association; 2003. *Stormwater Best Management Practice Handbook Construction.*

Department of Transportation; October 2001, *Field Guide to Construction Site Dewatering* CTSW-RT-01-010

Irvine Ranch Water District; 2004. *Rules and Regulations for Water, Sewer, Recycled Water, and Natural Treatment System Service.*

Montana Department of Transportation;2003.*Stormwater Best Management Practices*

Orange County Sanitation District; 1998. *Ordinance No. OCSD-01*

Washington State Department of Ecology; 2003. *Stormwater Management Manual for Eastern Washington.* Final Draft.

# Evaporation/Percolation Ponds

### Considerations

- Volume of water to be accommodated (minimum 1,000 gal/day) and space available to site a pond
- Cost $0.05/gal to $0.073/gal
- May require discharge to land permit if pond is unlined
- Monitoring may be required if discharge to land permit is required.
- Pond must be operated so that no water leaves the construction site
- If water remains in the pond for greater than 72 hours the Vector Control Agency may have monitoring requirements.
- Wildlife may use the pond for habitat.

### Description

Evaporation/percolation ponds are ponds with no outlet, which settle out the suspended solids, heavy metals and hydrocarbons that are adsorbed to sediment. Water is retained on-site in ponds to promote evaporation rather than transported or discharged off-site. These ponds may also be used to infiltrate some water or to store for reuse on the site (i.e. used for dust control). The evaporation of water can result in a decrease in the volume of water that must be managed.

Should the evaporation pond be unlined, the infiltration of water in the pond may be considered a discharge to land and consequently may be required to comply with the Statewide General Waste Discharge Requirements for Discharges to Land with a Low Threat to Water Quality (Order No. 2003-0003-DWQ). This "Land Discharge General Permit" is a statewide permit that specifies restrictions for various types of discharges to land. For certain types of discharges within the Santa Ana Region (see Table 3 on Page 1), dischargers may be able to obtain a waiver from the need to obtain coverage under the Land Discharge General permit, but only after submitting a Report of Waste Discharge for consideration by Regional Board staff. (Refer to the Discharge to Land Fact Sheet for additional information).

### Advantages

- Coverage under the San Diego Creek/Newport Bay De Minimus Permit or Region Wide General De Minimus Permit is not required as no discharge to surface waters will occur.
- If existing ponds (such as golf course lakes or neighborhood artificial water features) are available near the site, then such ponds may be utilized as evaporation/peroclation ponds with agreement of the owner.

### Limitations

- Some concern about safety when constructed where there is public access or where pond would be deemed an attractive nuisance for trespassers (especially children). Therefore, access may need to be limited through fencing or other barriers.
- Requires a relatively large land area available on which to locate the pond.

# Evaporation/Percolation Ponds

- Mosquito breeding is possible (ponded water should evaporate or percolate within 72 hours to limit this possibility).

- Ponds cannot be placed on steep unstable slopes.

- Evaporation rates slower (not expected to result in significant water loss) during cold weather. Clayey soils will limit infiltration.

- High selenium concentrations in ponds may limit or eliminate the ability to use such ponds as the high selenium may negatively affect wildlife using the ponds (especially birds). While this BMP is secondary to reducing the amount of groundwater that must be pumped in the first place, use of evaporation/percolation ponds is preferable to discharging into receiving waters where there may be active wildlife uses. Measures such as mylar streamers or similar should be considered to discourage birds or other wildlife from using the pond.

- May lead to concentrations of pollutants within the sediments or remaining waters of the pond that could then require special disposal (e.g., as "hazardous waste") depending upon their concentration. Any special disposal required for wastewaters or sediments would add costs to the use of the pond. Determination of pollutant concentration in bottom sediments or unevaporated waters would require testing prior to disposal, adding additional costs to the use of the pond.

- Ponds require maintenance to ensure that habitat attracting animals (e.g., wetland vegetation) is not present as the concentrating of pollutants may be harmful to wildlife utilizing the pond.

- Pond would need to be constructed large enough to provide adequate freeboard, so that in the event of precipitation, the pond did not overflow (potentially creating flood hazards or water quality problems in downstream receiving waters).

- If the pond is unlined, operation may require coverage under Land Discharge General Permit or a waiver from the Santa Ana Regional Board (a Report of Waste Discharge for the discharge must be filed for consideration – see Discharge to Land fact sheet).

- An annual fee will be required if coverage under the Land Discharge General Permit is required. The annual fee for most short-term operations ranges from $800 to $3,400.

### Requirements

- If the pond is unlined, dischargers must seek coverage under the Land Discharge General Permit or seek a waiver from the Santa Ana Regional Board ( refer to the Discharge to Land fact sheet).

- Capture volume determined by volume of water expected (with allowance for adequate freeboard in the event of a storm).

- Water depth not to exceed approximately 4 feet.

- Surface water area should be maximized to extent possible to allow for maximum evaporation/infiltration.

# Evaporation/Percolation Ponds

- Freeboard of at least 1 foot should be maintained at all times. A spillway should be included in the event rainfall into the evaporation pond causes it to overflow.
- Stormwater flows should not be directed to the pond.
- Liner may be used to limit infiltration outflow volumes in areas where ground water contamination may be a concern, in locations where the seasonal water table may adversely impact the pond, or to avoid triggering requirements under the land disposal general permit.
- Evaporation and mean annual precipitation rates are available from National Oceanic and Atmospheric Administration (NOAA) or Western Region Climate Center. For Orange County, the annual precipitation is approximately 13 inches, and the average annual evaporation rate is approximately 56 inches.
- In areas where ponded water cannot be evaporated/infiltrated within 72 hours, mechanical aeration can be used along with a monitoring program for mosquito larvae.
- Fencing or other barriers may need to be used to limit access by the public or trespassers.
- If sediment or water that doesn't evaporate/infiltrate accumulates in the pond, the sediment or excess water must be properly managed, either dispersed on-site or properly disposed.

*Maintenance*

The amount of maintenance required for an evaporation/percolation pond is highly dependent on local regulatory agencies, particular health and vector control agencies. These agencies are often extremely concerned about the potential for mosquito breeding that may occur if standing water occurs for a period longer than 72 hours.

Typical maintenance activities and frequencies include:

- Monitoring for vectors
- Maintaining mechanical aeration equipment (if used)
- Clearance of any wildlife-attracting habitat.
- Disposal of accumulated sediments or unevaporated water (including pre-disposal testing if necessary).
- Alternate disposal of waters if needed to maintain adequate freeboard.
- Repair of perimeter barriers (e.g., fencing).

## Cost Considerations

*Construction Cost*

Evaporation/percolation ponds are a relatively inexpensive practice for disposal of groundwater-related discharges. Construction cost data for an evaporation/percolation pond is estimated to be $50/cubic yard. For every gal/day of discharge an area of 9.1 square feet is required in the southern California area to evaporate. (Based on 56 in/yr evaporation).

*Maintenance Cost*

For ponds, the annual cost of routine maintenance, (e.g. inspections and sediment disposal) has typically been estimated at about 3 to 5 percent of the construction cost.

Table 7 below shows typical construction and operation/maintenance costs for an evaporation/percolation pond. This table includes the construction of the pond, water transport to the pond, and maintenance of the pond. This example does not include costs associated with the disposal of any accumulated sediment and/or unevaporated water.

**Table 7 Evaporation Pond Construction/Operation Cost**

| Item | Cost | Gallons to be Disposed in one year | Cost / Gallon |
|---|---|---|---|
| Construction Cost | $50,000 | 1,000,000 | $ 0.05 |
| Sediment Testing | $500 | As needed | - |
| Permit Fee | $3,400 | Annual Fee | - |
| Maintenance Cost | $2,000 | Annually | - |
| Operator and Water Truck/Pump (if necessary) | $200 | Per 9,000 gallons | $0.022 |
| | Total Cost | for 1,000,000 gallons | $78,100 |

### References and Sources of Additional Information

Boise Public Works; 2000. *Stormwater Operations and Maintenance*.

California Stormwater Quality Association; 2003. *Stormwater Best Management Practice Handbook Construction*.

Department of Transportation; October 2001, *Field Guide to Construction Site Dewatering* CTSW-RT-01-010

Washington State Department of Ecology; 2003. *Stormwater Management Manual for Eastern Washington*. Final Draft.

# Offsite Transportation

## Considerations

- Volume of water to be accommodated (2,500 gal/day to 4,500 gal/day per truck)
- Transport / Disposal Contractor fee ($0.129/gal to $0.136/gal)
- Testing is required to determine water quality prior to transport.

### Description

A licensed transport, storage and disposal (TSD) contractor may remove, transport, store and dispose or treat (as necessary) water in lieu of other disposal methods or in the event that wastewater has concentrations of pollutants that prohibit its discharge to land under the Waste Discharge Requirements. Offsite transportation would be most applicable to discharges classified as "hazardous" or "designated" as defined in Title 22 CCR, Section 66261 and California Water Code Section 13173. Presented in Table 8 are the constituent concentrations listed in Title 22 CCR, Section 66261. The TSD contractor provides treatment of water and/or hazardous waste disposal of the water.

**Table 8 Title 22 Constituent Concentrations to be Considered Hazardous**

| Constituent | Maximum for any single composite sample (mg/L) |
|---|---|
| Antimony | 0.1 |
| Arsenic | 0.5 |
| Barium | 7.6 |
| Beryllium | 0.01 |
| Cadmium | 0.05 |
| Chromium (total) | 0.33 |
| Lead | 0.15 |
| Mercury | 0.009 |
| Nickel | 1 |
| Selenium | 0.16 |
| Silver | 0.3 |
| Thallium | 0.02 |
| Zinc | 70 |

# Offsite Transportation

### Advantages

- Can be used to dispose of water that is generally prohibited by Waste Discharge Permits.
- Coverage under the San Diego Creek/Newport Bay De Minimus Permit or Region Wide General De Minimus Permit is not required as no discharge to surface waters will occur.
- May be better than other BMPs if time savings (from not obtaining other permits that may be required) are needed to avoid potential liquidated damages.

### Limitations

- Offsite disposal can be prohibitively expensive.
- Testing is required to determine the levels of constituents, as shown in Table 9, within the water. Contact the TSD contractor to determine their requirements.
- Onsite collection and storage may be required to hold water prior to transport.
- Volume of water may be prohibitive—meaning that more water may need to be transported than can reasonably be transported using trucks and storage tanks.
- Storage tanks may be required to hold water until transportation vehicles arrive.

### Requirements

- Offsite disposal is typically appropriate for water with pollutants that cannot be discharged elsewhere. This option would be most feasible for water that contains pollutants in concentrations that make it more cost effective to transport the water offsite. This option may also be used where there is not enough area for on-site disposal or where on-site conditions are not otherwise amenable to disposal.
- Offsite transportation of water requires a fee to the licensed contractor. Testing of the water is required to determine the levels of pollution prior to pickup and transportation of the water. Hazardous waste manifest may be required, and manifests or other documentation should be provided to verify that water was properly disposed of.
- If applicable, the project EIR should be consulted to ensure that the additional truck trips from the site were included in the mitigation analysis and requirements.
- If necessary, onsite storage tanks may be required to hold water while awaiting vehicular transportation.

### Cost Considerations

Offsite disposal is one of the more costly options for disposal of water. Estimated cost for truck disposal is $450 to truck to a location within a 30-mile radius. Offsite disposal would be most cost effective for disposal of a small amount of water, at a rate of one to two trucks per day. One truck holds between 2,500 and 4,000 gallons of water. This is equivalent to a flow rate of 0.004 cfs to 0.01 cfs (see Equation 1). Costs presented in Table 9 do not include the cost of testing or the costs for on-site storage containers that may be required to store water between truck trips. It does include the cost for the operator to pump water into truck, transportation offsite, treatment and/or hazardous waste disposal.

# Offsite Transportation

Equation 1

$$\frac{2{,}500\ \text{gal/day}}{646{,}000\ \text{(gal/day)/cfs}} = 0.004\ \text{cfs}$$

**Table 9 Cost Example**

| Item | Cost to Dispose | Gallons to be Disposed | Cost / Gallon |
|---|---|---|---|
| Pumping | $50/hr | 9,000 | $0.0055 / gal |
| Operator and Transport Fees | $450 | Per 3,500 gallons | $ 0.13 / gal |
| Water Quality Testing | $600 | Prior to discharge | - |
| | Total Cost | for 1,000,000 gallons | $134,700 |

## References and Sources of Additional Information


Department of Transportation; October 2001, *Field Guide to Construction Site Dewatering* CTSW-RT-01-010

Washington State Department of Ecology; 2003. *Stormwater Management Manual for Eastern Washington*. Final Draft.

# Re-Injection

### Considerations

- Cost $0.22/gal to $1.27/gal
- Project site conditions must be suitable to accommodate the re-injection

### Description

Re-injection consists of taking the extracted groundwater and directing it back into the groundwater basin at a location away from the dewatering operation. Re-injection typically involves collecting the water from the dewatering site, potentially storing it and then re-injecting it into the soil at the same project site. Note that this fact sheet is limited to situations where groundwater will be re-injected within the same project area from which it is extracted, since this will not trigger Regional Board permitting requirements. Extracted groundwater that is re-injected at a location away (outside of) the project area will require a Regional Board permit and compliance with Basin Plan groundwater limits.

### Advantages

- Coverage under the San Diego Creek/Newport Bay De Minimus Permit or Region Wide General De Minimus Permit is not required as there is no discharge to surface waters.
- No monitoring or Regional Board permits are required provided that the re-injection occurs within the same project site from where groundwater is extracted.

### Limitations

- The rate at which water could be re-injected will need to be determined and may be limited. Data from shallow wells on the former El Toro Air Station indicate that injection wells would accept about 5 gallons per minute into the shallow aquifer.
- Multiple injection wells may be required, and onsite collection and storage equipment may also be required to hold water prior to re-injection.
- Due to the noise generated by the pumps, there may be restrictions on hours of operation.
- Re-injection at a location outside of the project site will require a Regional Board permit and compliance with Basin Plan groundwater limits, as well as associated monitoring and permit fees.

### Requirements

- Onsite storage of extracted groundwater or multiple injection wells will likely be required, as re-injection rates seldom equal the rate of extraction.
- Discharge of any wastes to ground waters is prohibited. The groundwater extraction/re-injection operation must be conducted to prevent any potential construction site pollutants from entering the re-injection stream.

# Re-Injection

- The project site will need sufficient space to accommodate at least one and likely multiple injection wells, related equipment, and temporary extracted groundwater storage.

### Cost Considerations

Cost considerations associated with this BMP include equipment needed to extract and then re-inject the water, including pumps, piping and injection wells. Temporary water storage may also be needed. Re-injection is generally a more expensive volume reduction practice due to the cost associated with installing a re-injection well system. Table 10 below provides an example of typical expenses to expect when implementing a re-injection BMP for a project with a total anticipated discharge of 1,000,000 gallons.

**Table 10 Re-injection Cost Example**

| Item | Cost | Gallons to be Disposed | Cost/Gallon | |
|---|---|---|---|---|
| Pumping | $50/hr | 300 gallons / hr | $0.167 / gal | |
| Injection Well Construction | $55,000 each | 7,200 gallons / day (continuous operation at 5 gal/min) | $ 7.64/ gal/day | |
| | Total Cost | for 1,000,000 gallons (using 1 well) | $221,700 | (140 days) |
| | Total Cost | for 1,000,000 gallons (using 10 wells) | $716,700 | (14 days) |
| | Total Cost | for 1,000,000 gallons (using 20 wells) | $1,266,700 | (7 days) |

### References and Sources of Additional Information


Todd, David Keith; 1980 *Groundwater Hydrology.*

City of Irvine; January 2004, *Dewatering Alternatives Project Study Report*.

**ATTACHMENT B**
**Decision Tree**
**For Compliance with De Minimus Discharge Permit Requirements**
**Within Orange County (Santa Ana Region)**