Riverside County

<u>Whitewater River Region</u>

Stormwater Quality Best Management Practice

Design Handbook for Low Impact Development

Riverside County Flood Control and Water Conservation District

1995 Market Street

Riverside CA 92501

June 2014

This page intentionally left blank

to facilitate double-sided printing

Table of Contents

This page intentionally left blank

to facilitate double-sided printing

1.0 Introduction

The purpose of this Riverside County Whitewater River Region Stormwater Quality Best Management Practice Design Handbook for Low Impact Development (Handbook) is to provide selection and design guidance for stormwater Best Management Practices (BMPs) for Priority Development Projects (PDPs) within the Whitewater River Region of Riverside County while meeting the goals of Low Impact Development (LID), where feasible. LID in the Whitewater River Region seeks to control runoff and pollutants close to their source, but has a slightly different approach than in areas with more annual rainfall. The *Whitewater River Region Water Quality Management Plan Guidance Document* (June 2014, and revised January 2015) (WWR WQMP) defines LID/site design BMPs as:

> Activities or programs to educate the public or provide low cost non-physical solutions, as well as facility design or practices aimed at reducing urban runoff, increasing infiltration, reducing pollutant transport mechanisms, and minimizing the difference between pre- and post-development urban runoff. **More simply, LID/Site Design BMPs promote retention and/or feature a natural treatment mechanism to address a site's Potential Pollutants of Concern.**

The overall majority of PDPs within the Whitewater River Region have historically been, and continue to be subject to local onsite retention requirements (see Section 6.1, Table 4 - Local Land Use Authority Onsite Retention Requirements). In the past, these local requirements were implemented to address downstream impacts; more recently, these requirements have been noted for their ability to meet the goals of LID.

Important Note: If your project is located in an area subject to a local onsite retention requirement, then the retention facility utilized to address your project's runoff will be sized and designed to meet the requirements of the local land use authority, and per Section 3.5.1.2 of the WWR WQMP, additional LID/site design and/or treatment control BMPs will not be required, the LID/site design measurable goal described in Section 3.5.1.1 of the WWR WQMP will have been met, and this Handbook will not be applicable to your project. **Details for sizing and design of retention facilities to comply with local ordinance can be obtained from the applicable local land use authority.** If your project is not subject to local land use authority onsite retention requirement, then you must select LID/site design BMPs and/or treatment control BMPs to address Potential Project Pollutants of Concern (PPPC), and Hydrologic Conditions of Concern (HCOC) if necessary.

This Handbook provides guidance for:

- Selecting LID/site design BMPs to effectively address the PPPC generated by the project, while meeting the LID/site design measurable goal described in Section 3.5.1.1 of the WWR WQMP;
- Selection of site layout features to decrease the amount of impervious surface; and
- Sizing of selected LID/site design BMPs to treat the required water quality design volume (V_{BMP}) or water quality design flow rate (Q_{BMP}).

1.1 Limitations

Use of this Handbook is contingent upon acceptance of the limitations described in this section.

The Whitewater River Region has a desert climate where water is a precious resource. Local land use authorities and water districts in the region promote water conservation through a combination of ordinances and policies that require desert appropriate, low water use landscaping.

When selecting LID/site design BMPs for implementation as part of a PDP, thoughtful consideration of the BMP's requirement for supplemental water is essential. Many of the LID/site design BMPs in this Handbook are fully compatible with desert appropriate, low water use landscaping requirements. However, some LID/site design BMPs, such as *Landscaped Swales* and *Landscaped Filter Strips*, rely on dense vegetation to stabilize soils and promote infiltration and treatment of runoff. It is recommended that use of LID/site design BMPs that rely on dense vegetation be limited to areas of the site where the BMP can be integrated into landscaping planned for other purposes/uses and planned to feature BMP-compatible vegetation.

This Handbook is provided as a resource and guide to assist in the selection, sizing, and design of BMPs to be incorporated into PDPs in the Whitewater River Region of Riverside County. Due to the varied nature of PDPs, this Handbook does not address development-specific or site-specific conditions and situations. Accordingly, it is the responsibility of the user of this Handbook to seek and retain the services of a Civil Engineer registered in the State of California with professional knowledge, skills, and capabilities relative to development and stormwater quality control for selection and design of BMPs incorporated into the project. Users of this Handbook assume all responsibility and liability, including safety and pollution liability, for their selection, sizing, design, implementation, and use of structural BMPs incorporated into projects; this responsibility continues throughout the life of the project.

Users of this Handbook shall immediately discontinue its use for a project if the user identifies site-specific conditions or site-specific situations where use of the Handbook may not yield a safe and/or environmentally sustainable outcome. Users that identify errors or omissions in this Handbook that affect its use shall immediately notify the Riverside County Flood Control and Water Conservation District (District). The notification shall be made in writing, shall identify the specifics of the error or omission, shall provide contact information for the individual making the notification, and shall be delivered to the District's Watershed Protection Section in Riverside via Certified Mail addressed as follows:

Riverside County Flood Control and Water Conservation District
Attention: Watershed Protection Division
1995 Market Street
Riverside, CA 92501

2.0 Tributary Drainage Area

BMPs are required at locations where runoff from developed areas leaves the project site. These locations are referred to as ***discharge points***. The selection, sizing, and design of BMPs are a function of the area that contributes runoff to each ***discharge point***. The area contributing runoff to a ***discharge point*** is referred to as the ***tributary drainage area***. Identifying how a PDP drains is important to the selection, sizing, and design of BMPs, and involves these key steps:

- Identify the ***discharge points*** where runoff from developed areas leaves the project site.
- Identify the ***tributary drainage area*** for each ***discharge point***.
- Calculate the ***impervious ratio*** for the ***tributary drainage area***.

Minimizing the amount of impervious surface within a ***tributary drainage area*** will help to reduce the overall runoff that the BMP will need to address. In addition, incorporating site design BMP concepts such as ***Self-Retaining Areas*** (SRA) or ***Self-Treating Areas*** (STA) can reduce the tributary area that is required to be addressed by a BMP.

2.1 Self-Retaining Areas

A Self-Retaining Area (SRA) is an area within a PDP that has been designed to capture and retain the volume of runoff requiring treatment from that area. **SRAs can be removed from the total *tributary drainage area* used to size BMPs**; however, this project area should also be accounted for later as project area which counts towards the LID/site design measurable goal described in Section 3.5.1.1 of the WWR WQMP**.**

Site design features that may qualify as SRAs include, but are not limited to:

- Landscaped Areas
- Pervious Pavement
- Ponds
- Fountains
- Retention Rooftops

SRAs can provide "dual use" functionality as stormwater retention areas and project amenities. Dual uses include features that provide aesthetic value in the landscape design or that provide for outdoor recreation such as playing fields or parks.

When SRAs have no adjacent ***tributary drainage area***, then documentation to support the area functioning as a SRA is simple: one only needs to demonstrate that the SRA will retain the equivalent of 0.40 inches of rainfall. As an example, consider a landscaped area within a concrete patio. If the concrete patio slopes away from the landscaped area, and the landscaped area is level and has a finished grade that is one inch below the surrounding concrete (i.e., to prevent irrigation runoff), then that inch of inset landscaping will more than retain 0.40 inches of rainfall, and the area qualifies as a SRA.

SRAs can serve an expanded role when the SRA is designed to retain not only the design storm falling directly on the SRA, but also the runoff from the ***tributary drainage areas*** that are adjacent to the SRA. To qualify as a SRA, the ***tributary drainage area*** must be adjacent to the SRA, and must have a footprint area no larger than the footprint of the SRA where the rainfall and runoff will be retained.

When SRAs have an adjacent ***tributary drainage area***, the documentation to support the area functioning as a SRA does not need to be complex: one needs to demonstrate that the SRA will retain the equivalent of 0.40 inches of rainfall onto the SRA and its tributary area. As an example, consider the previous example of a landscaped area within a concrete patio. This time, however, consider that the concrete patio has been sloped *towards* the landscaped area, and the concrete patio has a surface area equal to that of the landscaped area. Again, if the landscaped area is level and has a finished grade that is one inch below the surrounding concrete (i.e., to prevent irrigation runoff), then that one inch of inset landscaping will more than retain the equivalent of 0.80 inches of rainfall, including 0.40 inches falling directly on the landscaping and another 0.40 inches of falling on the patio and flowing into the landscaping.

SRAs are not recommended for soils which are not expected to be freely draining, so as not to create vector or nuisance conditions. Compaction within SRAs should be minimized or avoided entirely where possible.

2.2 Self-Treating Areas

A Self-Treating Area (STA) is an area within a PDP site that does not drain to a BMP, but drains directly offsite or to the MS4, rather than having its runoff comingle with runoff from the project's impervious surfaces. These areas will not require specialized operation and maintenance procedures, and can typically be maintained with normal landscape maintenance. **STAs can also be removed from the total *tributary drainage area* used to size BMPs**; however, this project area should also be accounted for later as project area which counts towards the LID/site design measurable goal described in Section 3.5.1.1 of the WWR WQMP**.**

Site design features that may qualify as STAs include, but are not limited to:

- Conserved Natural Spaces
- Undeveloped Areas
- Landscaped Areas

STAs must direct their drainage away from any other areas of the site that must be addressed by LID/site design or treatment control BMPs, otherwise they will add to the volume or rate of runoff to be addressed by those BMPs. To avoid co-mingling with flows, flows from STAs must be directed offsite, such as into the storm drain system.

Landscaped areas must be predominately pervious if they are to qualify as an STA. Impervious landscape features within an STA must be limited to 10% or less of the total area of the STA, and the impervious areas must not be directly connected to areas offsite via flow across the site boundary or via flow to onsite or offsite storm drains. Disconnecting an impervious area requires that the runoff from the impervious area sheet flow in a non-erosive manner across a pervious area that is as wide as the contributing impervious surface.

3.0 BMP Selection

Selection of LID/site design BMPs for a PDP is a function of three primary factors:

- The potential pollutants generated by land uses within the BMP's ***tributary drainage area***;
- The sensitivity of receiving waters to the project's potential pollutants; and
- The types of available BMPs and their effectiveness at addressing the project's potential pollutants.

3.1 Identifying Project Pollutants of Concern

The types of potential pollutants which may be generated by a project's ***tributary drainage area*** to a BMP are a function of the land uses and legacy pollutants (if any) within the ***tributary drainage area***. Table 1 identifies potential pollutants based on land use. The potential pollutant(s) from all land uses within a project's ***tributary drainage area*** are herein referred to as ***Potential Project Pollutants of Concern (PPOC)***.

Table 1: Potential Pollutants Generated by Land Use Type

Type of Development (Land Use)	**General Pollutant Categories**						
	Sediment/ Turbidity	Nutrients	Toxic Organic Compounds	Trash & Debris	Bacteria & Viruses (also: Pathogens)	Oil & Grease	Heavy Metals
Detached Residential Development	P	P	N	P	P	P	N
Attached Residential Development	P	P	N	P	P	P[2]	N
Commercial/ Industrial Development	P	P[1]	P[5]	P	P[3]	P	P[6]
Automotive Repair Shops	N	N	P[4,5]	P	N	P	P
Restaurants	N	N	N	P	P	P	N
Hillside Development	P	P	N	P	P	P	N
Parking Lots	P	P[1]	P[4]	P	P	P	P
Retail Gasoline Outlets	N	N	P[4]	P	N	P	P

Abbreviations: P = Potential N = Not potential

Notes:

(1) A potential Pollutant if non-native landscaping exists or is proposed onsite; otherwise not expected.
(2) A potential Pollutant if the project includes uncovered parking areas; otherwise not expected.
(3) A potential Pollutant if land use involves food or animal waste products.
(4) Specifically, petroleum hydrocarbons.
(5) Specifically, solvents; however, this Pollutant is not expected at commercial office or commercial retail sites, unless said retail is vehicle related.
(6) A potential Pollutant if the project includes outdoor storage or metal roofs; otherwise not expected.

3.2 Identifying Receiving Water Impairments

Receiving waters which will receive runoff from each ***discharge point*** on a PDP site must be identified. Once the receiving waters have been identified, the ***PPPOC*** causing impairment of the receiving water must be identified (if any). Information on how to identify a PDP site's receiving waters, and also how to identify the ***PPPOC*** causing impairment is presented in Sections 3.1 and 3.3 (respectively) of the WWR WQMP. ***PPPOC***identified as causing impairment of receiving waters (if any) are herein referred to as ***Pollutants Impairing Receiving Waters***.

3.3 Addressing Potential Project Pollutants of Concern

BMPs must be deployed at each of the PDP's ***discharge points*** to address all of the ***PPPOC*** identified based on land uses and legacy pollutants (if any) within the BMP's ***tributary drainage area***. Where a ***PPPOC*** is the same as a ***Pollutant Impairing Receiving Waters***, then that pollutant must be addressed with an LID/site design BMP (or combination of BMPs) that is capable of high or medium effectiveness for addressing that pollutant.

3.4 BMP Pollutant Removal Effectiveness

There are different types of LID/site design BMPs available to address pollutants in urban runoff, and key to their selection is their effectiveness at addressing ***PPPOC***.

Table 2 is used to identify candidate BMPs to address ***PPPOC*** for each ***tributary drainage area*** on a project site.

- Select a BMP or combination of BMPs to address all ***PPPOC*** for each ***tributary drainage area*** on a project.
- When a ***PPPOC*** is the same as a ***Pollutant Impairing Receiving Waters***, the selected candidate BMP or combination of BMPs must have a pollutant removal effectiveness rating of medium or high for that ***PPPOC***.

Table 2: BMP Selection Matrix Based Upon Pollutant of Concern Removal Efficiency [(1)]

(Sources: Riverside County Flood Control & Water Conservation District's *Design Handbook for Low Impact Development Best Management Practices* (September 2011), the Orange County *Technical Guidance Document for Water Quality Management Plans* (May 19, 2011), and the Caltrans *Treatment BMP Technology Reports* (April 2010 and April 2008)

Pollutant of Concern	Landscape Swale[2]	Landscape Strip[2]	Biofiltration (with Underdrain)[2, 3]	Extended Detention Basin[2]	Sand Filter Basin[2]	Infiltration Basin[2]	Infiltration Trench[2]	Permeable Pavement[2]	Bioretention (w/o Underdrain)[2, 3]	Other BMPs Including Proprietary BMPs[4, 6]
Sediment & Turbidity	M	M	H	M	H	H	H	H	H	Varies by Product[5]
Nutrients	L/M	L/M	M	L/M	L/M	H	H	H	H	
Toxic Organic Compounds	M/H	M/H	M/H	L	L/M	H	H	H	H	
Trash & Debris	L	L	H	H	H	H	H	L	H	
Bacteria & Viruses (also: Pathogens)	L	M	H	L	M	H	H	H	H	
Oil & Grease	M	M	H	M	H	H	H	H	H	
Heavy Metals	M	M/H	M/H	L/M	M	H	H	H	H	

<u>Abbreviations:</u>

L: Low removal efficiency M: Medium removal efficiency H: High removal efficiency

<u>Notes:</u>

(1) Periodic performance assessment and updating of the guidance provided by this table may be necessary.

(2) Expected performance when designed in accordance with the most current edition of the document, *Riverside County, Whitewater River Region Stormwater Quality Best Management Practice Design Handbook for Low Impact Development*.

(3) Performance dependent upon design which includes implementation of thick vegetative cover. Local water conservation and/or landscaping requirements should be considered; approval is based on the discretion of the local land use authority.

(4) Includes proprietary stormwater treatment devices as listed in the CASQA Stormwater Best Management Practices Handbooks, other stormwater treatment BMPs not specifically listed in this WQMP (including proprietary filters, hydrodynamic separators, inserts, etc.), or newly developed/emerging stormwater treatment technologies.

(5) Expected performance should be based on evaluation of the unit processes used by the BMP and available BMP testing data. Approval is based on the discretion of the local land use authority.

(6) When used for primary treatment as opposed to pre-treatment, requires site-specific approval by the local land use authority.

3.5 Final BMP Selection

Final LID/site design BMP selection begins with a list of candidate BMPs developed using Table 2, and then requires narrowing the list of candidate BMPs based on several factors, including:

- Site-specific constraints and aesthetics

- Water conservation landscaping requirements
- Vector control requirements
- Operation and maintenance considerations
- Costs
- Class V Well status

<u>Site-Specific Constraints and Aesthetics</u>

Generally, infiltration BMPs have advantages over other types of BMPs, including reduction of runoff rate and volume, as well as (in most cases) full treatment of ***PPPOC***. It is recognized however that infiltration may not be feasible on some sites, such as those with high groundwater, low infiltration rates, or located within 50 feet horizontally from a water supply well.

BMPs will ultimately need to be integrated into the overall site plan and, therefore must be selected with consideration of site constraints such as setbacks and proximity to offsite features, and aesthetics.

<u>Water Conservation Landscaping</u>

Water is a precious resource in the Whitewater River Region's desert environment. Local land use authorities and water districts in the region promote water conservation through a combination of ordinances and policies that require desert appropriate, low water use landscaping. Table 5 in Section 6.2 of this Handbook identifies the local land use authorities in the region and their respective water conservation landscape ordinance. When selecting LID/site design BMPs for implementation, thoughtful consideration of the BMP's requirement for supplemental water and compliance with the applicable water conservation landscape ordinance is essential.

Many of the LID/site design BMPs in this Handbook are compatible with desert appropriate, low water use landscaping requirements. When incorporating vegetation into a BMP, Table 5 in Section 6.2 of this Handbook should be consulted to identify whether the local land use authority has adopted a specific desert plant palette. Where the local land use authority has not adopted a plant palette, the plant palettes in Table 5 may provide a starting point for selection of plants (subject to approval by the local land use authority).

Some LID/site design BMPs, such as *Landscaped Swales* and *Landscaped Filter Strips*, rely on dense vegetation to stabilize soils and promote infiltration and treatment of runoff. Use of LID/site design BMPs that rely on dense vegetation should be limited to areas of the site where the BMP can be integrated into landscaping planned for other purposes/uses and planned to feature BMP-compatible vegetation.

<u>Vector Control</u>

The occasional wet environment associated with a stormwater BMP has the potential to create conditions conducive to the breeding of vectors (e.g., mosquitos). This Handbook addresses vector issues through pre-design practices (e.g., infiltration testing) and design requirements (e.g., drawdown times limited to 48 hours). Vector issues are also addressed through BMP maintenance requirements described in the BMP fact sheets contained in the Appendix of this Handbook.

For further guidance on vector control as it relates to stormwater BMPs, refer to the document, *Best Management Practices for Mosquito Control in California* (California Department of Public Health and the Mosquito and Vector Control Association of California, July 2012). The document can be downloaded at http://www.cdph.ca.gov/HealthInfo/discond/Documents/BMPforMosquitoControl07-12.pdf (URL Verified June 2014).

Operation and Maintenance

The operation of maintenance requirements of candidate BMPs must be considered, because all BMPs require maintenance and maintenance continues for the life of the project. Operation and maintenance requirements for BMPs are presented in the BMP facts sheets in the Appendix of this Handbook.

Costs

Costs associated with BMPs includes capital costs and on-going operation and maintenance costs. It's important to consider life-cycle costs when determining the true costs of BMPs.

Class V Well Status

The United States Environmental Protection Agency (USEPA) is required by the Federal Safe Drinking Water Act to protect underground sources of drinking water; as part of its program, the USEPA regulates Class V stormwater drainage wells. The USEPA defines Class V stormwater drainage wells as follows:

> "Class V storm water drainage wells manage surface water runoff (rainwater or snow melt) by placing it below the ground surface. They are typically shallow disposal systems designed to infiltrate storm water runoff below the ground surface. Storm water drainage wells may have a variety of designs and may be referred to by other names including dry wells, bored wells, and infiltration galleries. The names may be misleading so it is important to note that **a Class V well by definition is any bored, drilled, or driven shaft, or dug hole that is deeper than its widest surface dimension, or an improved sinkhole, or a subsurface fluid distribution system (an infiltration system with piping to enhance infiltration capabilities)**." [Emphasis added]

The USEPA also provides guidance on what it does not consider to be a Class V stormwater drainage well:

> "Some types of infiltration systems do not meet the definition of Class V storm water drainage wells. Infiltration trenches are generally larger at their widest surface point than they are deep, and they do not contain any perforated pipes or drain tiles to distribute and/or facilitate subsurface fluid infiltration. Surface impoundments do not include dug, drilled, or driven shafts."

Generally, it is preferable to select BMPs that are not regulated as a Class V stormwater drainage wells. When a selected BMP includes <u>any</u> feature that falls within the definition of a Class V stormwater drainage well, the BMP <u>must</u> meet the requirements of the Class V Well Permitting Authority, which in the Whitewater River Region is USEPA Region 9. USEPA Region 9 requires that they be contacted before construction begins, and that information about the proposed Class V stormwater drainage well be submitted to the USEPA Region 9 inventory. This process provides USEPA Region 9 the opportunity to place requirements on the proposed Class V stormwater drainage well. Information from USEPA Region 9, including links to the on-line Class V stormwater drainage well inventory system can be found here: http://www.epa.gov/region09/water/groundwater/uic-classv.html.

3.6 Basis for BMP Design

Once the list of candidate BMPs is narrowed based on the factors described above, the design basis for the selected BMP needs to be determined. To simplify the design procedures contained in this Handbook, the design basis for the BMPs featured are either volume based or flow based. Volume based BMPs are designed based on a treatment volume and flow based BMPs are designed based on a treatment flow rate. Table 3 identifies the BMPs included in this Handbook and whether they are volume based or flow based. The design basis of the BMP will be used to calculate the selected BMPs sizing requirement (See Section 4).

Some BMPs, such as sand filters, bioretention, and biofiltration, can be designed based on a combination of volume based and flow based criteria; however, this is more complicated, and beyond the scope of the simplified design procedures in this Handbook. Use of combined design basis is subject to approval of the local land use authority.

Table 3: Design Basis for BMPs

Structural BMP	Design Basis
Landscaped Swales	Q_{BMP}
Landscaped Filter Strips	
Biofiltration (w/ Underdrain)	V_{BMP}
Extended Detention Basin	
Sand Filter Basin	
Infiltration Basin	
Infiltration Trench	
Permeable Pavement	
Bioretention (w/o Underdrain)	
Other BMPs	Q_{BMP} or V_{BMP} on Case-Specific Basis as Approved by the Local Land Use Authority

4.0 BMP Sizing

After selection of either volume based or flow based BMPs, the next step is to calculate the V_{BMP} or Q_{BMP} design criteria, which are based on three main factors:

- The ***tributary drainage area*** to the BMP (See Section 2)
- The degree of imperviousness in the ***tributary drainage area***, expressed as the ***Impervious Ratio***.
- The climatic factor, expressed as a Unit Capture Volume for volume based BMPs (0.40 Inches), and as a Rainfall Intensity for flow based BMPs (0.2 Inches per Hour). These factors are built into Worksheets 1 and 2 below for calculation of V_{BMP} and Q_{BMP}.

The procedures in this Handbook for calculation of V_{BMP} and Q_{BMP} are specific to the Whitewater River Region, and to the requirements of the WWR MS4 Permit issued by the California Regional Water Quality Control Board, Colorado River Basin Region (Regional Board) to the Riverside County Flood Control and Water Conservation District, the County of Riverside, Coachella Valley Water District, and incorporated cities of Riverside County within the Whitewater River Basin (Permittees); use of alternative procedures for calculation of V_{BMP} and Q_{BMP} are not permitted for the purpose of sizing BMPs for compliance with WWR MS4 Permit requirements for PDPs in the Whitewater River Region. Additionally, these BMP sizing procedures should **not** be used when a project is subject to a local land use authority onsite retention ordinance (see Section 6.1 below).

4.1 Impervious Ratio

The pervious portion of a site, such as, but not limited to landscaped or natural areas, is where rainfall has the opportunity to infiltrate into the ground. Impervious surfaces, such as, but not limited to rooftops and traditionally paved parking lots, are where rainfall has no opportunity to infiltrate and immediately becomes surface runoff. The imperviousness of the ***tributary drainage area*** to a BMP is a significant factor in how big a selected BMP will need to be.

The imperviousness of the ***tributary drainage area*** to a BMP is expressed in terms of the drainage area's ***Impervious Ratio*** (I_f). The ***Impervious Ratio*** is the ratio of the area of impervious surfaces in the ***tributary drainage area*** to the total ***tributary drainage area***.

$$I_f = \frac{Impervious\ Area}{Total\ Area}$$

As a reminder, if applicable, SRAs and/or STAs (see Sections 2.1 and 2.2 above) should be subtracted from the total ***tributary drainage area*** before calculating for I_f. This subtracted project area should be accounted for later, however, as it counts towards the LID/site design measurable goal, as described in Section 3.5.1.1 of the WWR WQMP.

Following are examples of impervious surfaces:

- Concrete or Asphalt Paving (Traditional, Impervious Paving)
- Roofs

- Grouted or Gapless Paving Blocks
- Compacted Soil (Compacted to greater than 85% and not landscaped)

Following are examples of pervious surfaces:

- Paver Systems with Sand/Gravel Filled Gaps (Designed to Promote Infiltration)
- Pervious Concrete and Porous Asphalt Paving
- Turf Blocks
- Gravel or Class 2 Permeable Base (Compacted to 85% or less)
- Ornamentally Landscaped Areas
- Decomposed Granite Surfaces (Compacted to 85% or less)
- Natural Soils (Never Graded or Compacted)
- Engineered Soil (Compacted to 85% or less)

4.2 Drawdown Times

Volume based BMPs are usually associated with a required drawdown time; this is the amount of time the design volume takes to pass through the effective storage area of the BMP. The drawdown times specified in the design of BMPs in this Handbook provide adequate contact or detention time for treatment, while not creating vector or other nuisance issues. They are also a factor in determining the climatic factor (0.40 inches) for volume based design. It is important to abide by the drawdown time requirements stated in the fact sheet for each specific BMP. Drawdown times typically range from 24 to 48 hours, but should not exceed 48 hours. Longer drawdown times require site-specific continuous simulation modeling to determine Unit Basin Storage Volumes based on the longer inter-event periods: accordingly, longer drawdown times may only be used with the approval of the local land use authority. When longer drawdown times are used, the BMP design worksheets in this Handbook may not apply.

4.2.1 Maximum Depth

The infiltration rate of the soils where the BMP will be placed will determine the maximum depth of volume based infiltration BMPs; the infiltration rate will govern depth in which the BMP can still drawdown within 48 hours. This is calculated by applying a safety factor to the infiltration rate to achieve the design infiltration rate. The safety factor applied is based on the type of information known about the soils and the type of infiltration testing performed. The design infiltration rate is then multiplied by the maximum drawdown time of 48 hours. The value calculated will be the maximum effective depth of storage in the BMP. Refer to the infiltration BMPfactsheets for specifics on maximum depth calculation procedures and minimum safety factors to be used.

4.3 V_{BMP} Calculation for Volume Based Design

Volume based BMPs are designed to treat a Unit Basin Storage Volume such that over time, 80% of all runoff will be captured and treated, consistent with the requirements of the WWR MS4 Permit. This may seem like a large volume on a per-event basis; however, the 80% capture requirement only requires capturing relatively small volumes because small storms over time collectively produce more runoff volume than the infrequent, larger storms. For more information on this phenomena, refer to Section 5.5 of the *Stormwater Best Management Practice Handbook – New Development and Redevelopment* (CASQA, 2003).

For the Whitewater River Region, the unit basin storage volume is equivalent to **0.40 inches** of rainfall, with a drawdown time of 48 hours or less. In order to meet WWR MS4 Permit requirements for PDPs, volume based BMPs must be designed to treat the unit basin storage volume.

Use Worksheet 1 to calculate V_{BMP} for the ***tributary drainage area*** to each BMP.

4.4 Q_{BMP} Calculation for Flow Based Design

Flow based BMPs are designed to treat the maximum rate of runoff produced from a rainfall intensity of **0.2 inches per hour**, for each hour of a rain event, and is consistent with the requirements of the WWR MS4 Permit.

Use Worksheet 2 to calculate Q_{BMP} for the ***tributary drainage area*** to each BMP.

Worksheet 1 – This worksheet is available on the District's Municipal Stormwater Management Program page for developers at: www.rcflood.org/NPDES/developers.aspx

Whitewater Watershed BMP Design Volume, V_{BMP} (Rev. 06-2014)		Legend:	Required Entries Calculated Cells
Company Name		**Date**	
Designed by		**County/City Case No**	
Company Project Number/Name			
Drainage Area Number/Name			
Enter the Area Tributary to this Feature (A_{TRIB})		A_{TRIB} =	acres
Determine the Impervious Area Ratio			
Determine the Impervious Area within A_{TRIB} (A_{IMP})		A_{IMP} =	acres
Calculate Impervious Area Ratio (I_f) $I_f = A_{IMP}/A_{TRIB}$		I_f =	
Calculate the composite Runoff Coefficient, C for the BMP Tributary Area			
Use the following equation based on the WEF/ASCE Method $C_{BMP} = 0.858I_f^3 - 0.78I_f^2 + 0.774I_f + 0.04$		C_{BMP} =	
Determine Design Storage Volume, V_{BMP}			
Calculate V_U, the 80% Unit Storage Volume $V_U = 0.40 \times C_{BMP}$		V_u =	(in*ac)/ac
Calculate the design storage volume of the BMP, V_{BMP}. $V_{BMP}\ (ft^3) = \dfrac{V_U\ (in\text{-}ac/ac) \times A_T\ (ac) \times 43{,}560\ (ft^2/ac)}{12\ (in/ft)}$		V_{BMP} =	ft^3
Notes:			

Worksheet 2 – This worksheet is available on the District's Municipal Stormwater Management Program page for developers at: www.rcflood.org/NPDES/developers.aspx

Whitewater Watershed

BMP Design Flow Rate, Q_{BMP} (Rev. 06-2014)

Legend: Required Entries / Calculated Cells

Company Name		**Date**	
Designed by		**County/City Case No**	
Company Project Number/Name			
Drainage Area Number/Name			

Enter the Area Tributary to this Feature (A_{TRIB}) A_{TRIB} = ______ acres

Determine the Impervious Area Ratio

Determine the Impervious Area within A_{TRIB} (A_{IMP}) A_{IMP} = ______ acres

Calculate Impervious Area Ratio (I_f) I_f = ______

$I_f = A_{IMP}/A_{TRIB}$

Calculate the composite Runoff Coefficient, C for the BMP Tributary Area

Use the following equation based on the WEF/ASCE Method

$C_{BMP} = 0.858 I_f^3 - 0.78 I_f^2 + 0.774 I_f + 0.04$ C_{BMP} = ______

BMP Design Flow Rate

$Q_{BMP} = C_{BMP} \times I \times A_{TRIB}$ Q_{BMP} = ______ ft^3/s

I = Design Rainfall Intensity, 0.2 in/hr

Notes:

This page intentionally left blank

to facilitate double-sided printing

5.0 BMP Fact Sheets and Design Worksheets

This Handbook includes factsheets and worksheets to assist in design of BMPs (See Appendix A). Factsheets provides a description of a BMP; worksheets facilitates BMP sizing and design calculations. The worksheets are available as functional Microsoft® Excel® worksheets; their use provides the minimum required size for a BMP based on the V_{BMP} or Q_{BMP} calculated according to Section 4 of this Handbook.

The design procedures included in this Handbook are based on the *Design Handbook for Low Impact Development Best Management Practices* (September 2011) and on the *Riverside County Whitewater River Region BMP Design Handbook Update Technical Memorandum* (May 2014). BMP factsheets and worksheets are provided for the following BMPs:

- Landscaped Swale;
- Landscaped Filter Strip;
- Biofiltration (with Underdrain);
- Extended Detention Basin;
- Sand Filter Basin;
- Infiltration Basin;
- Infiltration Trench;
- Permeable Pavement; and
- Bioretention (without Underdrain).

→ For portability, the fact sheets for each BMP, and calculation worksheets for sizing and documenting the design of these BMPs are provided as separate downloadable files on the District's Municipal Stormwater Management Program page for developers at: www.rcflood.org/NPDES/developers.aspx

This page intentionally left blank

to facilitate double-sided printing

6.0 Resources

6.1 Local Land Use Authority Onsite Retention Requirements

If your project is located in an area subject to a local onsite retention requirement (see Table 4 below), then the retention facility utilized to address your project's runoff will be sized and designed to meet the requirements of the local land use authority, and per Section 3.5.1.2 of the WWR WQMP, additional LID/site design and/or treatment control BMPs will not be required; the LID/site design measurable goal described in Section 3.5.1.1 of the WWR WQMP will have been met, and this Handbook will not be applicable to your project. **Details for sizing and design of retention facilities to comply with local ordinances can be obtained from the applicable local land use authority.** If your project is not subject to an onsite retention requirement, then you must select LID/site design BMPs and/or treatment control BMPs to address PPPOC, and HCOCs if necessary.

Table 4: Local Land Use Authority Onsite Retention Requirements

Local Land Use Authority	Requires Onsite Retention	Requirement	Ordinance
Banning	X	100% retention of 100yr, 3hr storm event	City Ordinance #1415
Cathedral City	X	100% retention of 100yr, 3hr storm event	Municipal Code - Title 8 Section 8.24.070
Coachella	X	100% retention of 100yr, 24hr storm event	City Ordinance #1014
Desert Hot Springs	X	100% retention of 100yr, 24hr storm event	City Ordinance #1997-03 Section 13.08.100
Indian Wells			
Indio	X	100% retention of 100yr, 24hr storm event	Code of Ordinances - Title XV: Land Usage 162.140
La Quinta	X	100% retention of 100yr, 24hr storm event	Municipal Code - Title 13 Section 13.24.120
Palm Desert	X	100% retention of 100yr, 24hr storm event	Municipal Code - Title 26 Section 26.49.060
Palm Springs	X	Difference between most conservative 100yr storm event in pre and post condition	Municipal Code - 9.60.030 (18) & (19)(A)
Rancho Mirage	X	Projects greater than or equal to one acre and located north of the Whitewater River retain the worst case duration 100yr storm	Municipal Code - Title 13 Section 13.05.010
Riverside County			

6.2 Local Land Use Authority Standards and Ordinances

Local land use authorities in the Whitewater River Region have in place ordinances, standards, and guidelines that can influence the selection and design of LID/site design BMPs. Table 5 identifies these ordinances, standards, and guidelines. Check with the local land use authority for additional requirements.

Table 5: Local Land Use Authority Standards and Ordinances

Local Land use Authority	Retention Basin Standard	Drywell Standard	Water Conservation Landscaping Ordinance	Desert Plant Palette
Banning			X[1]	
Cathedral City			X[1,2]	
Coachella	X	X	X[1,2]	
Desert Hot Springs		X	X[1]	X
Indian Wells			X[1,2]	
Indio	X	X	X[1,2]	
La Quinta		X	X[1,2]	
Palm Desert		X	X[1]	X
Palm Springs		X	X[1]	X
Rancho Mirage	X	X	X[1,2]	
Riverside County	X	X	X[1]	X
Coachella Valley Water District			X[2]	X

[1]Water Conservation Riverside County Landscaping Ordinance (859.2)

[2]Coachella Valley Water District Water Conservation Landscape Ordinance No. 1302.1

6.3 Local Land Use Authority Resource Links

The following links are provided to assist users of this Handbook in locating information relative to the selection and design of LID/site design BMPs in the Whitewater River Region. The links provided were verified in June 2014. The dynamic nature of the Internet may result in resources being moved or deleted and henceforth unavailable via the links provided. Furthermore, the information referenced may be superseded. Therefore, it is the responsibility of the user of this Handbook to contact the local land use authority to identify the specific requirements applicable to the project.

City of Banning: http://www.ci.banning.ca.us/index.aspx?nid=21

Cathedral City: http://www.cathedralcity.gov/index.aspx?page=478

City of Coachella: http://www.coachella.org/departments/public-works-department

City of Desert Hot Springs: http://www.cityofdhs.org/NPDES___PM-10

City of Indian Wells: http://www.cityofindianwells.org/cityhall/depts/pw.asp#.U07s_fmzHz8

City of Indio: http://www.indio.org/index.aspx?page=252

City of La Quinta: http://www.la-quinta.org/Index.aspx?page=147

City of Palm Desert: http://www.cityofpalmdesert.org/Index.aspx?page=205

City of Palm Springs: http://www.ci.palm-springs.ca.us/index.aspx?page=86

City of Rancho Mirage: http://www.ranchomirageca.gov/public-works

Riverside County: http://planning.rctlma.org/DevelopmentProcess/DesignGuidelines.aspx

Riverside County Flood Control District: http://rcflood.org/NPDES/WhitewaterWS.aspx

Coachella Valley Water District: http://www.cvwd.org

This page intentionally left blank

to facilitate double-sided printing

7.0 References

Riverside County Flood Control and Water Conservation District, September 2011. *Design Handbook for Low Impact Development Best Management Practices*

Riverside County Flood Control and Water Conservation District, June 2009. *Whitewater River Region Stormwater Quality Best Management Practice Design Handbook*

California Stormwater Quality Association, January 2003. *Stormwater Best Management Practice Handbook for New Development and Redevelopment*

Bay Area Stormwater Management Agencies Association, May 2003. *Using Site Design Techniques to Meet Development Standards for Stormwater Quality, A Companion Document to Start at the Source*

This page intentionally left blank

to facilitate double-sided printing

Appendix A – BMP Factsheets

→ For portability, these BMP factsheets, as well as calculation worksheets for sizing and documenting the design of these BMPs, are provided as separate downloadable files on the District's Municipal Stormwater Management Program page for developers at:

www.rcflood.org/NPDES/developers.aspx

Note: The Whitewater River Region has a desert climate where water is a precious resource. Local land use authorities and water districts in the region promote water conservation through a combination of ordinances and policies that require desert appropriate, low water use landscaping.

When selecting LID/site design BMPs for implementation as part of a PDP, thoughtful consideration of the BMP requirements for supplemental water is essential. Many of the LID/site design BMPs in this Handbook section are fully compatible with desert appropriate, low water use landscaping requirements. However, some LID/site design BMPs rely on dense vegetation to stabilize soils and promote infiltration and treatment of runoff. It is recommended that use of LID/site design BMPs that rely on dense vegetation be limited to areas of the site where the BMP can be integrated into landscaping planned for other purposes/uses and planned to feature BMP-compatible vegetation.

This page intentionally left blank

to facilitate double-sided printing

A.1 Landscaped Swales

Treatment Mechanisms	Infiltration, Evapotranspiration, Evaporation, Biofiltration
Maximum Drainage Area	This BMP is intended to be integrated into a project's landscaped area, typically, in conjunction with other BMPs.
Other Names	Vegetated Swales

A.1.1 General

A landscaped swale is a wide, shallow, landscaped channel that treats stormwater runoff as it is slowly conveyed into a downstream system. These swales have very shallow slopes in order to allow maximum contact time with the landscaping. The depth of water of the design flow should be less than the height of the landscaping cover. Contact with landscaping improves water quality by plant uptake of pollutants, removal of sediment, and an increase in infiltration. Overall, the effectiveness of a landscaped swale is limited. If utilized, it is recommended for use in combination with other BMPs.

A.1.2 Siting Considerations

This BMP is not appropriate for industrial sites or locations where spills occur. Important factors to consider when using this BMP include: natural channelization should be avoided to maintain this BMP's effectiveness, large areas must be divided and treated with multiple swales, thick cover is required to function properly, impractical for steep topography, and not effective with high-flow velocities.

Important Note: Local water conservation and/or landscaping requirements should be considered when planning for these facilities, as their design calls for implementation of thick vegetative cover. It is recommended that this BMP only be used in areas where thick vegetation or landscaping was already going to be incorporated into the project as part of the design.

A.1.3 Landscaped Swale Design Criteria

Table 1: Landscaped Swale Design Criteria

Design Parameter	Unit	Design Criteria
Design Flow	cfs	Q_{BMP}
Minimum bottom width	ft	2 ft
Maximum channel side slope	H:V	3:1
Minimum slope in flow direction	%	0.2 (provide underdrains for slopes < 0.5)
Maximum slope in flow direction	%	2.0 (provide grade-control checks for slopes >2.0)
Maximum flow velocity	ft/sec	1.0 (based on Manning n = 0.20)
Maximum depth of flow	inches	3 to 5 (1 inch below top of landscaping)
Minimum contact time	minutes	7
Minimum length	ft	Sufficient length to provide minimum contact time
Vegetation	-	Desert appropriate landscaping which yields thick cover, or approved equal
Landscaping height	inches	4 to 6 (mow/trim to maintain height)

Inspection and Maintenance Schedule

The landscaped swale area should be inspected for erosion, dead vegetation, soggy soils, or standing water. The use of fertilizers and pesticides on the plants inside the Landscaped Swale should be minimized.

Table 2: Inspection and Maintenance Schedule

Schedule	Activity
Ongoing	• Keep adjacent landscaped areas maintained. Remove clippings from landscape maintenance activities • Remove trash and debris • Remove accumulated sediment • Replace damaged landscaping and/or plants • Maintain vegetation to design height through periodic mowing and/or trimming
After storm events	• Inspect areas for ponding
Annually	• Inspect/clean any inlets and outlets

A.1.4 Landscaped Swale Design Procedure

1. Design Flow
   Use **Worksheet 2** - Design Procedure Form for Design Flow Rate, Q_{BMP}.

2. Swale Geometry
   a. Determine bottom width of swale (must be at least two feet).
   b. Determine side slopes (must not be steeper than 3:1; flatter is preferred).
   c. Determine flow direction slope (must be between 0.2% and 2%; provide underdrains for slopes less than 0.5% and provide grade control checks for slopes greater than 2.0%

3. Flow Velocity
   Maximum flow velocity should not exceed 1.0 ft/sec based on a Manning's n = 0.20

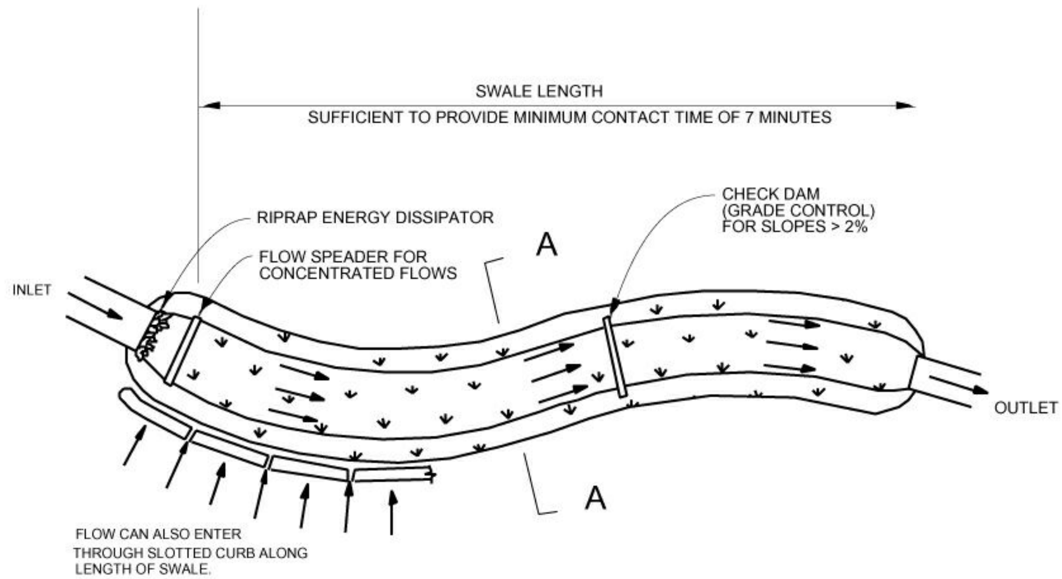
4. Flow Depth
   Maximum depth of flow should not exceed three to five inches based on a Manning n = 0.20

5. Swale Length
   Provide length in the flow direction sufficient to yield a minimum contact time of seven minutes.

   L = (7 min) x (flow velocity ft/s) x (60 sec/min)

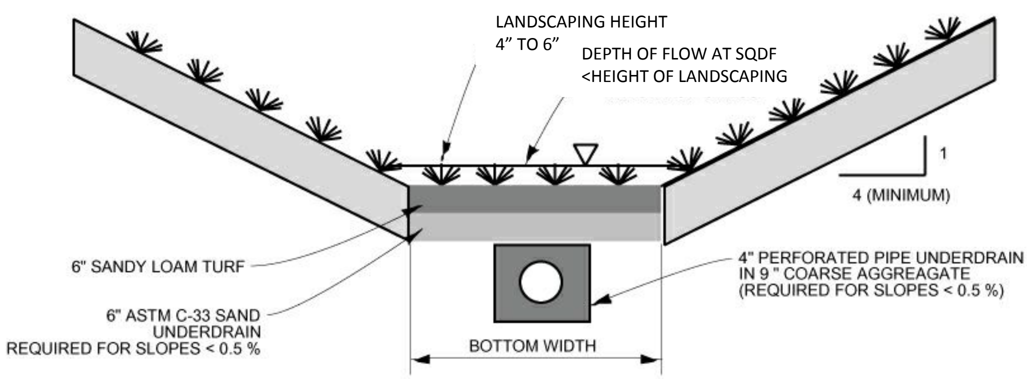
6. Vegetation
   Provide desert appropriate landscaping which yields full cover, or approved equal. Trim or mow to maintain height of four to six inches.

7. Provide sufficient flow depth for flood event flows to avoid flooding of critical areas or structures.



TRAPEZOIDAL LANDSCAPED SWALE SECTION

NOT TO SCALE



TRAPEZOIDAL LANDSCAPED SWALE SECTION

NOT TO SCALE

Figure 3: Landscaped Swale

Worksheet 3 – This worksheet is available at the District's Municipal Stormwater Management Program page for developers at: www.rcflood.org/NPDES/developers.aspx

Landscaped Swale - Design Procedure (Rev. 06-2014)	BMP ID	Legend:	Required Entries / Calculated Cells
Company Name:		Date:	
Designed by:		County/City Case No.:	

Design Volume

Enter the area tributary to this feature	A_{TRIB}=		acres
Enter Q_{BMP} determined from Section 4.4 of this Handbook	Q_{BMP}=		cfs

Landscaped Swale Geometry

Enter Swale bottom width (b), Minimum of 2 feet width.	b =		ft
Enter side slope of swale (must not be steeper than 3:1; flatter is preferred).	z =		
Enter flow directional slope (s)	s =		%

(must be between 0.2% and 2%; provide underdrains for slopes less than 0.5% and provide grade control checks for slopes greater than 2.0%)

Determining Swale Length

Enter design flow velocity (Manning n = 0.2), Max of 1.0 ft/s	v =		ft/s
Enter depth of flow (D) (should not exceed 3 to 5 inches)	D =		ft
Design length. L = 7(min) x (flow velocity, ft/sec) x 60	L =		ft

Describe landscaping:

Outfall Collection

Check Outfall Collection (check type used or describe "other")

_________ Grated Inlet
_________ Infiltration trench
_________ Underdrain
_________ Other :

Notes:

This page intentionally left blank

to facilitate double-sided printing

A.2 Landscaped Filter Strips

Treatment Mechanisms	Infiltration, Evapotranspiration, Evaporation, Biofiltration
Maximum Drainage Area	This BMP is intended to be integrated into a project's landscaped area in a distributed manner. Typically, contributing drainage areas to Landscaped Filter Strips range from less than one acre to a maximum of around five acres.
Other Names	Vegetated Filter Strip

A.2.1 General

Landscaped filter strips are uniformly graded areas of landscaping designed to treat sheet flow stormwater runoff. Pollutants are removed by filtration, and through settling of sediment and other solid particles as the design flow passes through (not over) the landscaping. Landscaped filter strips are usually as wide as their tributary area, and must be long enough in the flow direction to adequately treat the runoff. Concentrated flows are redistributed uniformly across the top of the strip with a level spreader. A landscaped swale, sand filter, or infiltration BMP is recommended in conjunction with a landscaped filter strip.

A.2.2 Siting Considerations

This BMP is not appropriate for industrial sites or locations where spills occur. Important factors to consider when using this BMP include: thick landscaped cover is required to function properly, and it is not effective if length and flow characteristics are not met.

Important Note: Local water conservation and/or landscaping requirements should be considered when planning for these facilities, as their design calls for implementation of thick vegetative cover. It is recommended that this BMP only be used in areas where thick vegetation or landscaping was already going to be incorporated into the project as part of the design.

A.2.3 Landscaped Filter Strip Design Criteria

Table 1: Landscaped Filter Strip Design Criteria

Design Parameter	Unit	Design Criteria
Design Flow	cfs	Q_{BMP}
Maximum Tributary Area	acres	5
Maximum Linear Unit Application Rate (q_a)	cfs/ft x width	0.005
Minimum Width (normal to flow)	ft	$(Q_{BMP}) / (q_a)$
Minimum Length (flow direction)	ft	15
Maximum Slope (flow direction)	%	4
Vegetation	-	Desert appropriate landscaping which yields thick cover, or approved equal
Minimum Landscaping Height	inches	2
Maximum Landscaping Height	inches	4 (typical) or as required to prevent debris build-up
Level Spreader	-	A level spreader must be applied to the flows before reaching the strip
Recommendation	-	This BMP is recommended in conjunction with a landscaped swale, sand filter, or infiltration BMP

Inspection and Maintenance Schedule

The landscaped filter strip area should be inspected for erosion, dead vegetation, soggy soils, or standing water. The use of fertilizers and pesticides on the plants inside the landscaped filter strip should be minimized.

Table 2: Inspection and Maintenance Schedule

Schedule	Activity
Ongoing	• Keep adjacent landscaped areas maintained. Remove clippings from landscape maintenance activities • Remove trash and debris • Remove accumulated sediment • Replace damaged landscaping and/or plants • Maintain vegetation to design height through periodic mowing and/or trimming
After storm events	• Inspect areas for ponding
Annually	• Inspect/clean any inlets and outlets

A.2.4 Landscaped Filter Strip Design Procedure

1. Design Flow

   Use **Worksheet 2** - Design Procedure Form for Design Flow Rate, Q_{BMP}.

2. Minimum Width

   Calculate minimum width of the landscaped filter strip (W_m) normal to flow direction:

   $$W_m = (Q_{BMP})/(q_a)$$

   $$W_m = (Q_{BMP})/0.005 \text{ cfs/ft (minimum)}$$

3. Minimum Length

   Length of the landscaped filter strip (L_m) in the direction of flow shall not be less than 15 feet.

   $$L_m = 15 \text{ feet (minimum)}$$

4. Maximum Slope

   Slope of the ground in the direction of flow shall not be greater than 4%.

5. Flow Distribution

   Incorporate a device at the upstream end of the filter strip to evenly distribute flows along the top width, such as slotted curbing, modular block porous pavement, or other spreader devices. Concentrated flow delivered to the filter strip must be distributed evenly by means of a level spreader of similar concept.

6. Vegetation

   Provide desert appropriate landscaping which yields full cover, or approved equal. Trim or mow landscaping to maintain height approximately between two and four inches.

7. Outflow Collection

Provide a means for outflow collection and conveyance (e.g. landscaped channel/swale, storm sewer, street gutter).

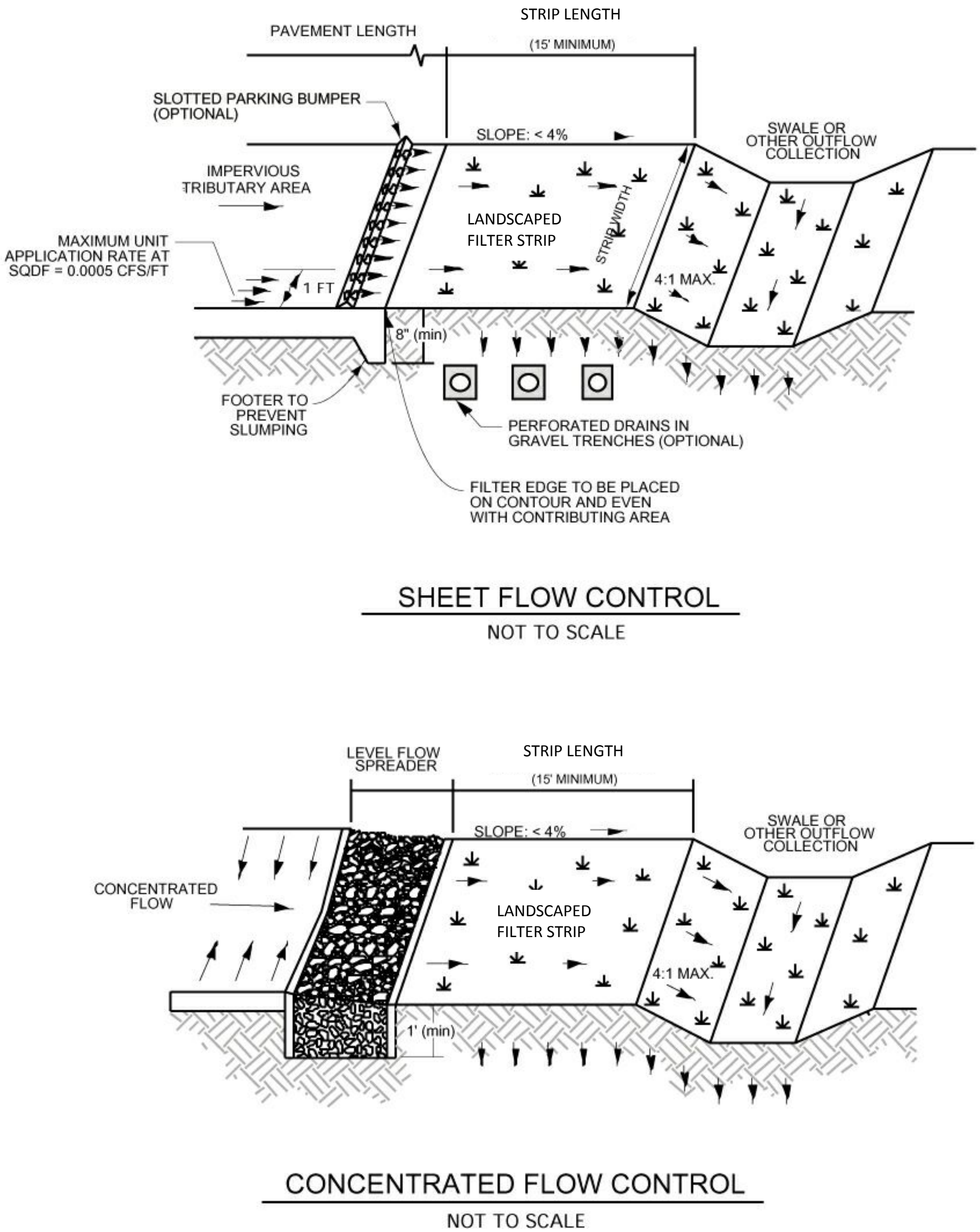


Figure 1: Landscaped Filter Strip

Worksheet 4 – This worksheet is available on the District's Municipal Stormwater Management Program page for developers at: www.rcflood.org/NPDES/developers.aspx

Landscaped Filter Strip - Design Procedure	BMP ID	Legend:	Required Entries / Calculated Cells
Company Name:		Date:	
Designed by:		County/City Case No.:	

Landscaped Filter Strip Design

Enter Q_{BMP} determined from Section 4.4 of this Handbook — Q_{BMP}= ______ cfs

Design Width $W_m = (Q_{BMP})/0.005$ cfs/ft — W_m = ______ ft

Landscaped Filter Strip Geometry

Design Length (L_m), Minimum of 15 feet — L_m = ______ ft

Design slope (S_o), Maximum slope 4% — S_o = ______

Flow Distribution:

______ Slotted curbing
______ Modular Block Porous Pavement
______ Level Spreader
______ Other :

Describe landscaping:

Outfall Collection

Check Outfall Collection (check type used or describe "other")

______ Landscaped Swale
______ Street Gutter
______ Storm Drain
______ Underdrain
______ Other :

Notes:

This page intentionally left blank

to facilitate double-sided printing

A.3 Bioretention Facilities

Treatment Mechanisms	Infiltration, Evapotranspiration, Evaporation, Biofiltration
Maximum Drainage Area	This BMP is intended to be integrated into a project's landscaped area in a distributed manner. Typically, contributing drainage areas to Bioretention Facilities range from less than one acre to a maximum of around 10 acres.
Other Names	Landscaped retention, Rain Garden, Bioretention Cell, Bioretention Basin, Biofiltration Basin, Landscaped Filter Basin, Porous Landscape Detention

A.3.1 General

Bioretention facilities are shallow, landscaped basins underlain by an engineered soil media. Healthy plant and biological activity in the root zone maintain and renew the macro-pore space in the soil and maximize plant uptake of pollutants and runoff. This keeps this BMP from becoming clogged, and allows more of the soil column to function as both a sponge (retaining water) and a highly effective and self-maintaining biofilter. In most cases, the bottom of a bioretention facility is unlined, which also provides an opportunity for infiltration to the extent the underlying onsite soil can accommodate. When the infiltration rate of the underlying soil is exceeded, fully biotreated flows are discharged via underdrains. Bioretention facilities, therefore, will inherently achieve the maximum feasible level of infiltration and evapotranspiration and achieve the minimum feasible (but highly biotreated) discharge to the storm drain system.

A.3.2 Siting Considerations

These facilities work best when they are designed in a relatively level area. Unlike other BMPs, bioretention facilities can be used in smaller landscaped spaces on the site, such as:

- Parking islands
- Medians
- Site entrances

Landscaped areas on the site (such as may otherwise be required through minimum landscaping ordinances), can often be designed as bioretention facilities. This can be accomplished by:

- Depressing landscaped areas below adjacent impervious surfaces, rather than elevating those areas;
- Grading the site to direct runoff from those impervious surfaces *into* the Bioretention facility, rather than away from the landscaping; and/or
- Sizing and designing the depressed landscaped area as a bioretention facility as described in this FactSheet.

Bioretention facilities should not be used downstream of areas where large amounts of sediment can clog the system. Placing a bioretention facility at the toe of a steep slope should also be avoided due to the potential for clogging the engineered soil media with erosion from the slope, as well as the potential for damaging the vegetation.

Important Note: Local water conservation and/or landscaping requirements should be considered when planning for these facilities, as effective landscaping cover is essential to their function. It is recommended that this BMP be used in areas where landscaping was already going to be incorporated into the project as part of the design.

A.3.3 Bioretention Facility Design Criteria

Table 1: Bioretention Facility Design Criteria

Design Parameter	Unit	Design Criteria
Design Flow	cfs	V_{BMP}
Maximum Tributary Area	acres	10
Minimum Width	ft	6
Maximum Side-Slope	-	4:1
Maximum Ponding Depth	inches	6
Minimum Side-Slope Width (Maximum Ponding Depth X Maximum Side-slope)	ft	2
Minimum Depth of Engineered Soil Media	inches	18
Minimum Depth of Gravel Layer	inches	12
Vegetation	-	Desert appropriate landscaping suitable for this BMP with 2-3 inch layer of mulch
Engineered Soil Media	-	85% mineral and 15% organic by volume. The mineral component must meet the range specified in Table 2 below; the organic component must be nitrogen stabilized compost

The recommended cross-section necessary for a bioretention facility includes:

- Landscaped area;
- 18” minimum depth of engineered soil media; and
- 12” minimum gravel layer depth with 6' perforated pipes (added flow control features such as orifice plates may be required to mitigate for HCOC conditions)

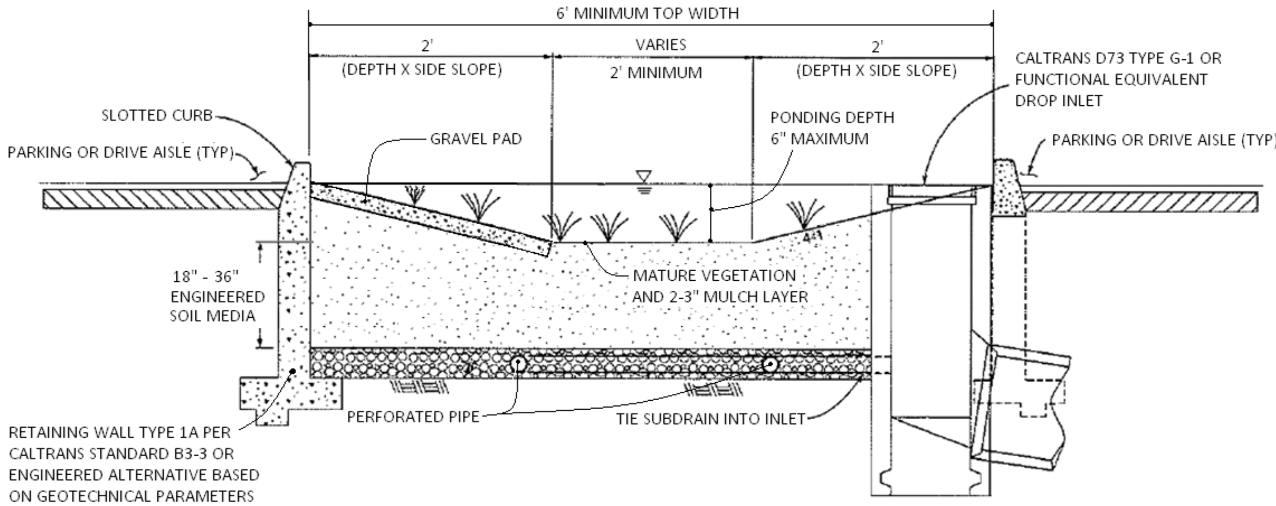


Figure 1: Standard Layout for Bioretention Facility

While the 18-inch minimum engineered soil media depth can be used in some cases, it is recommended to use 24 inches or a preferred 36 inches to provide an adequate root zone for the chosen plant palette. Such a design also provides for improved removal effectiveness for nutrients. The recommended ponding depth inside of a bioretention facility is six inches; measured from the flat bottom surface to the top of the water surface as shown in Figure 1.

Because this BMP is filled with an engineered soil media, pore space in the soil and gravel layer is assumed to provide storage volume. However, several considerations must be noted:

- Surcharge storage above the soil surface (six inches) is important to assure that design flows do not bypass the BMP when runoff exceeds the soil's absorption rate.
- In cases where the bioretention facility contains engineered soil media deeper than 36 inches, the pore space within the engineered soil media can only be counted to the 36 inch depth.
- A maximum of 30% pore space can be used for the soil media whereas a maximum of 40% pore space can be used for the gravel layer.

Engineered Soil Media Recommendations

The engineered soil media should be comprised of 85%mineral component and 15% organic component, by volume, drum mixed prior to placement. The mineral component should be a Class A sandy loam topsoil that meets the range specified in Table 2. The organic component should be nitrogen stabilized compost[1], such that nitrogen does not leach from the media.

[1] For more information on compost, visit the US Composting Council website at: http://compostingcouncil.org/

Table 2: Mineral Component Range Requirements

Percent Range	Component
70-80	Sand
15-20	Silt
5-10	Clay

Landscaping Requirements

Landscaping cover is important to minimize erosion and ensure that treatment occurs in the bioretention facility. The area should be designed for mature coverage throughout the bioretention facility using desert-appropriate landscaping. To prevent the BMP from being used as walkways, bioretention facilities are recommended to be planted with a combination of small trees, planted shrubs, and native landscaping. Landscaping should be native or ornamental; preferably ones that do not need to be mowed or irrigated. The application of fertilizers and pesticides should be minimal. To maintain oxygen levels for the vegetation and promote biodegradation, it is important that vegetation not be completely submerged for any extended period of time. Therefore, a maximum of six inches of ponded water should be used in the design to ensure that plants within the Bioretention Facility remain healthy.

A 2 to 3-inch layer of standard shredded aged hardwood mulch should be placed as the top layer inside the bioretention facility. The 6-inch ponding depth shown in Figure 1 above should be measured from the top surface of the 2 to 3-inch mulch layer.

Curb Cuts

To allow water to flow into the bioretention facility, one foot wide (minimum) curb cuts should be placed approximately every 10 feet around the perimeter of the bioretention facility. Figure 2 shows a curb cut in a bioretention facility. Curb cut flow lines must be at or above the V_{BMP} water surface level.

Figure 2: Curb Cut located in a Bioretention Facility

To reduce erosion, a gravel pad should be placed at each inlet point to the bioretention facility. The gravel should be one to 1.5-inch diameter in size. The gravel should overlap the curb cut opening a minimum of six inches. The gravel pad inside the bioretention facility should be flush with the finished surface at the curb cut and extend to the bottom of the slope.

In addition, place an apron of stone or concrete, a foot square or larger, inside each inlet to prevent vegetation from growing up and blocking the inlet. See Figure 3.

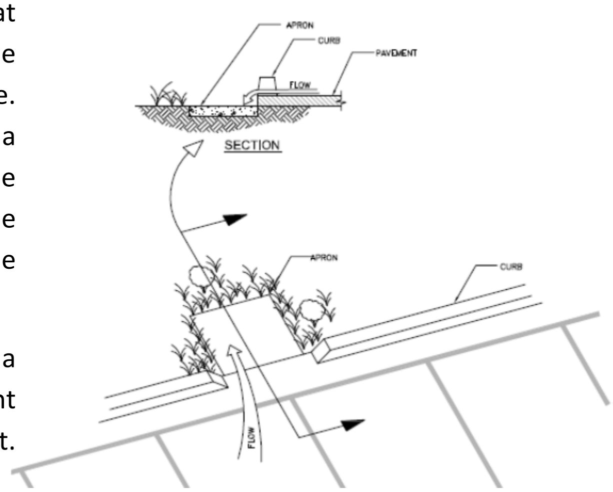


Figure 3: Apron located in a Bioretention Facility

Terracing the Bioretention Facility

It is recommended that bioretention facilities be level. In the event the facility site slopes and lacks proper design, water would fill the lowest point of the BMP and then discharge from the basin without being treated. To ensure that the water will be held within the bioretention facility on sloped sites, the BMP must be terraced with nonporous check dams to provide the required storage and treatment capacity.

The terraced version of this BMP should be used on non-flat sites with no more than a 3% slope. The surcharge depth cannot exceed 0.5 feet, and side slopes should not exceed 4:1. Table 3 below shows the spacing of the check dams, and slopes should be rounded up (i.e., 2.5% slope should use 10' spacing for check dams).

Table 3: Check Dam Spacing

6” Check Dam Spacing	
Slope	Spacing
1%	25'
2%	15'
3%	10'

Roof Runoff

Roof downspouts may be directed towards bioretention facilities. However, the downspouts must discharge onto a concrete splash block to protect the bioretention facility from erosion.

Retaining Walls

It is recommended that Retaining Wall Type 1A, per Caltrans Standard B3-3 or equivalent, be constructed around the entire perimeter of the bioretention facility. This practice will protect the sides of the bioretention facility from collapsing during construction and maintenance or from high service loads adjacent to the BMP. Where such service loads would not exist adjacent to the BMP, an engineered alternative may be used if signed by a licensed civil engineer.

Side Slope Requirements

Bioretention Facilities Requiring Side Slopes

The design should assure that the bioretention facility does not present a tripping hazard. Bioretention facilities proposed near pedestrian areas, such as areas parallel to parking spaces or along a walkway, must have a gentle slope to the bottom of the facility. Side slopes inside of a bioretention facility should be 4:1. A typical cross-section for the bioretention facility is shown in Figure 1.

Bioretention Facilities Not Requiring Side Slopes

Where cars park perpendicular to the bioretention facility, side slopes are not required. A six inch maximum drop may be used, and the bioretention facility must be planted with trees and shrubs to prevent pedestrian access. In this case, a curb is not placed around the bioretention facility, but wheel stops should be used to prevent vehicles from entering the bioretention retention facility, as shown in Figure 4.

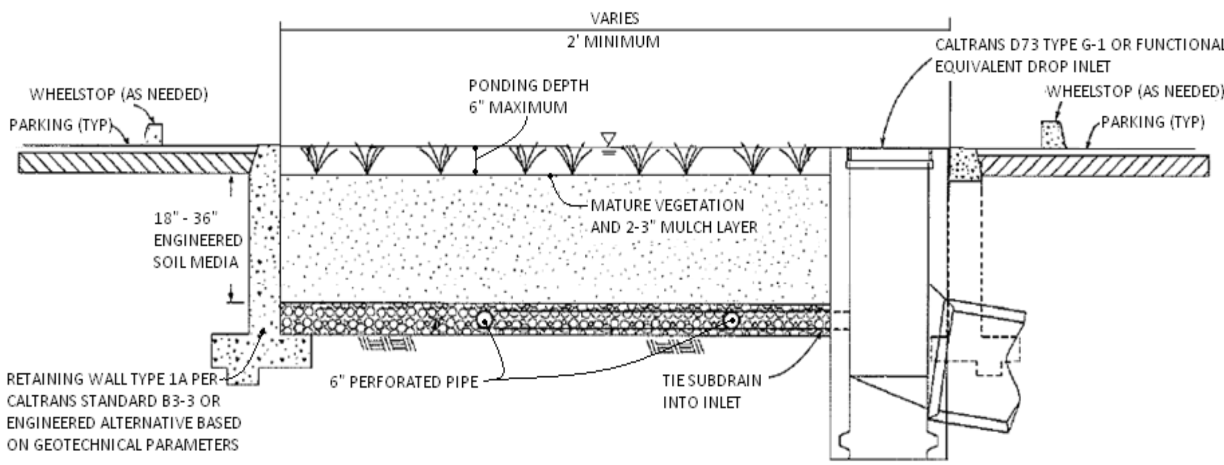


Figure 4: Bioretention Facility Not Requiring Slopes

Planter Boxes

Bioretention facilities can also be placed above ground as planter boxes. Planter boxes are recommended to have a minimum width of two feet, a maximum surcharge depth of six inches, and no side slopes are necessary. Planter boxes must be constructed so as to ensure that the top surface of the engineered soil media will remain level. This option may be constructed of concrete, brick, stone or other stable materials that will not warp or bend. Chemically treated wood or galvanized steel, which has the ability to contaminate stormwater, should not be used. Planter boxes should be lined with an impermeable liner on all sides, including the bottom. Due to the impermeable liner, the inside bottom of the planter box should be designed and constructed with a cross-fall, directing treated flows within the sub-drain layer toward the point where sub-drain exits the planter box, and sub-drains should be oriented with drain holes oriented down. These provisions will help avoid excessive stagnant water within the gravel underdrain layer. Similar to the in-ground bioretention facility versions, this BMP benefits from healthy plants and biological activity in the root zone. Planter boxes should be planted with appropriately selected desert-appropriate landscaping.

Figure 5: Planter Box

Overflow

An overflow route is needed in the bioretention facility design to bypass stored runoff from storm events larger than V_{BMP} or in the event of facility or sub-drain clogging. Overflow systems must connect to an acceptable discharge point, such as a downstream conveyance system as shown in Figure 1 and Figure 4. The inlet to the overflow structure should be elevated inside the bioretention facility to be flush with the ponding surface for the design volume (V_{BMP}) as shown in Figure 5. This will allow the design capture volume to be fully treated by the bioretention facility, and for larger events to safely be conveyed to downstream systems. The overflow inlet should not be located in the entrance of a bioretention facility, as shown in Figure 6.

Underdrain Gravel and Pipes

An underdrain gravel layer and pipes should be provided in accordance with Appendix C – Underdrain Guidelines.

Figure 6: Incorrect Placement of an Overflow Inlet.

Inspection and Maintenance Schedule

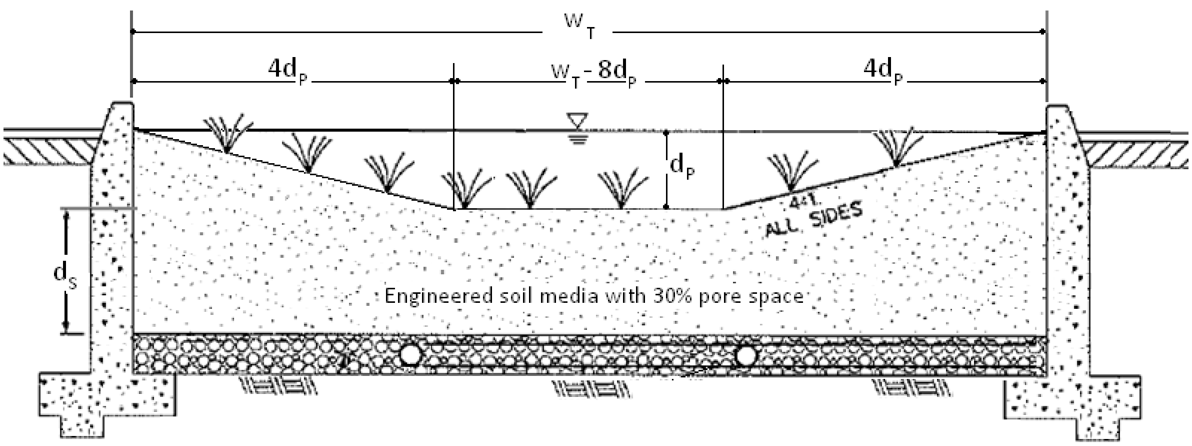
The bioretention facility area should be inspected for erosion, dead vegetation, soggy soils, or standing water. The use of fertilizers and pesticides on the plants inside the bioretention facility should be minimized.

Table 4: Inspection and Maintenance Schedule

Schedule	Activity
Ongoing	• Keep adjacent landscape areas maintained. Remove clippings from landscape maintenance activities. • Remove trash and debris. • Replace damaged landscaping and/or plants. • Replace surface mulch layer as needed to maintain a 2-3 inch soil cover.
After storm events	• Inspect areas for ponding.
Annually	• Inspect/clean inlets and outlets.

A.3.4 Bioretention Facility Design Procedure

1. Find the Design Volume, V_{BMP}.
   a) Enter the Tributary Drainage Area, A_{TRIB}.
   b) Enter the Design Volume, V_{BMP}, determined from **Worksheet 1** of this Handbook.
2. Select the type of design used. There are two types of bioretention facility designs: the standard design used for most project sites that include side slopes, and the modified design used when the BMP is located perpendicular to the parking spaces or with planter boxes that do not use side slopes.
3. Enter the depth of the engineered soil media, d_S. The minimum depth for the engineered soil media can be 18” in limited cases, but it is recommended to use 24” or a preferred 36” to provide an adequate root zone for the chosen plant palette.
4. Enter the top width of the bioretention facility.
5. Calculate the total effective depth, d_E, within the bioretention facility. The maximum allowable pore space of the soil media is 30% while the maximum allowable pore space for the gravel layer is 40%. Gravel layer deeper than 12” will only get credit for the pore space in the first 12”.



   a. For the design with side slopes the following equation should be used to determine the total effective depth. Where, d_P is the depth of ponding within the basin.

$$d_E(\text{ft}) = \frac{0.3 \text{ x } \left[\left(W_T(\text{ft}) \text{x } d_S(\text{ft})\right) + 4\left(d_P(\text{ft})\right)2\right] + 0.4 \text{ x } 1(\text{ft}) + d_P(\text{ft})[4d_P(\text{ft}) + \left(W_T(\text{ft}) - 8d_P(\text{ft})\right)]}{W_T(\text{ft})}$$

   This above equation can be simplified if the maximum ponding depth pf 0.5’ is used. The equation below is used on the worksheet to find the minimum area required for the bioretention facility.

$$d_E(\text{ft}) = \left(0.3 \text{ x } d_S(\text{ft}) + \ 0.4 \text{ x } 1(\text{ft})\right) - \left(\frac{0.7\ (\text{ft}^2)}{W_T(\text{ft})}\right) + 0.5(\text{ft})$$

b. For the design without side slopes the following equation should be used to determine the total effective depth:

$$d_E(ft) = d_P(ft) + [(0.3)x\ d_S(ft) + (0.4)x\ 1(ft)]$$

The equation below, using the maximum ponding depth of 0.5', is used on the worksheet to find the minimum area required for the bioretention facility:

$$d_E(ft) = 0.5(ft) + [(0.3)x\ d_S(ft) + (0.4)x\ 1(ft)]$$

6. Calculate the minimum surface area, A_M, required for the bioretention facility. This does not include the curb surrounding the bioretention facility or side slopes.

$$A_M(ft^2) = \frac{V_{BMP}(ft^3)}{d_E(ft)}$$

7. Enter the proposed surface area. This area should not be less than the minimum required surface area.
8. Verify that side slopes are no steeper than a ratio of 4:1 in the standard design, and are not required in the modified design.
9. Provide the diameter, minimum six inches, of the perforated underdrain used in the bioretention facility. See Appendix C for specific information regarding perforated pipes.
10. Provide the slope of the site around the bioretention facility, if used. The maximum slope is 3% for a standard design.
11. Provide the check dam spacing if the site around the bioretention facility is sloped.
12. Describe the landscaping used within the bioretention facility.

Worksheet 5 – This worksheet is available on the District's Municipal Stormwater Management Program page for developers at: www.rcflood.org/NPDES/developers.aspx

Bioretention Facility - Design Procedure (Rev. 06-2014)	BMP ID	Legend:	Required Entries	
			Calculated Cells	
Company Name:		Date:		
Designed by:		County/City Case No.:		

Design Volume

Enter the area tributary to this feature	A_{TRIB}=		acres
Enter V_{BMP} determined from Section 4.3 of this Handbook	V_{BMP}=		ft^3

Type of Bioretention Facility Design

- (●) Side slopes required (parallel to parking spaces or adjacent to walkways)
- ( ) No side slopes required (perpendicular to parking space or Planter Boxes)

Bioretention Facility Surface Area

Depth of Soil Filter Media Layer	d_S =		ft
Top Width of Bioretention Facility, excluding curb	w_T =		ft
Total Effective Depth, d_E $d_E = (0.3) \times d_S + (0.4) \times 1 - (0.7/w_T) + 0.5$	d_E =		ft
Minimum Surface Area, A_m $A_M\ (ft^2) = \dfrac{V_{BMP}\ (ft^3)}{d_E\ (ft)}$	A_M =		ft^2
Proposed Surface Area	A=		ft^2

Bioretention Facility Properties

Side Slopes in Bioretention Facility	z =		:1
Diameter of Underdrain			inches
Longitudinal Slope of Site (3% maximum)			%
6" Check Dam Spacing			feet
Describe Landscaping:			

Notes:

This page intentionally left blank

to facilitate double-sided printing

A.4 Extended Detention Basin BMP Factsheet

Treatment Mechanisms	Sedimentation, Infiltration, Biofiltration, Evapotranspiration, and Evaporation
Minimum Tributary Drainage Area	5 acres
Other Names	Enhanced Water Quality Basin

A.4.1 General

The Extended Detention Basin (EDB) is designed to detain the design volume of stormwater, V_{BMP}, and maximize opportunities for volume losses through infiltration, evaporation, evapotranspiration, and surface wetting. Additional pollutant removal is provided through sedimentation, in which pollutants can attach to sediment accumulated in the basin through the process of settling. Stormwater enters the EDB through a forebay where any trash, debris, and sediment accumulate for easy removal. Flows from the forebay enter the basin which is vegetated with desert appropriate native landscaping that enhance infiltration and evapotranspiration, and which is interspersed with gravel-filled trenches that help further enhance infiltration. Water that does not get infiltrated or evapotranspired is conveyed to the bottom stage of the basin. At the bottom stage of the basin, low or incidental dry weather flows will be treated through a sand filter and collected in a sub-drain structure. Any additional flows will be detained in the basin for an extended period by incorporating an outlet structure that is more restrictive than a traditional detention basin outlet. The restrictive outlet structure extends the drawdown time of the basin which further allows particles and associated pollutants to settle out before exiting the basin, while maximizing opportunities for additional incidental volume losses.

A.4.2 Siting Considerations

Soils

EDBs can be used with almost all soils and geology. However, pollutant removal effectiveness is greatly improved when the underlying soil permits at least some infiltration.

Tributary Drainage Area

EDBs should only be used where the tributary drainage area is at least 5 acres, since meeting the draw-down requirements (discussed below) for smaller areas would result in very small outlet orifice diameters which would be prone to clogging.

Proximity to Receiving Waters

All site runoff must be treated to the MEP with appropriate BMPs before being discharged into receiving waters; as such the EDB cannot be constructed in-line within receiving waters.

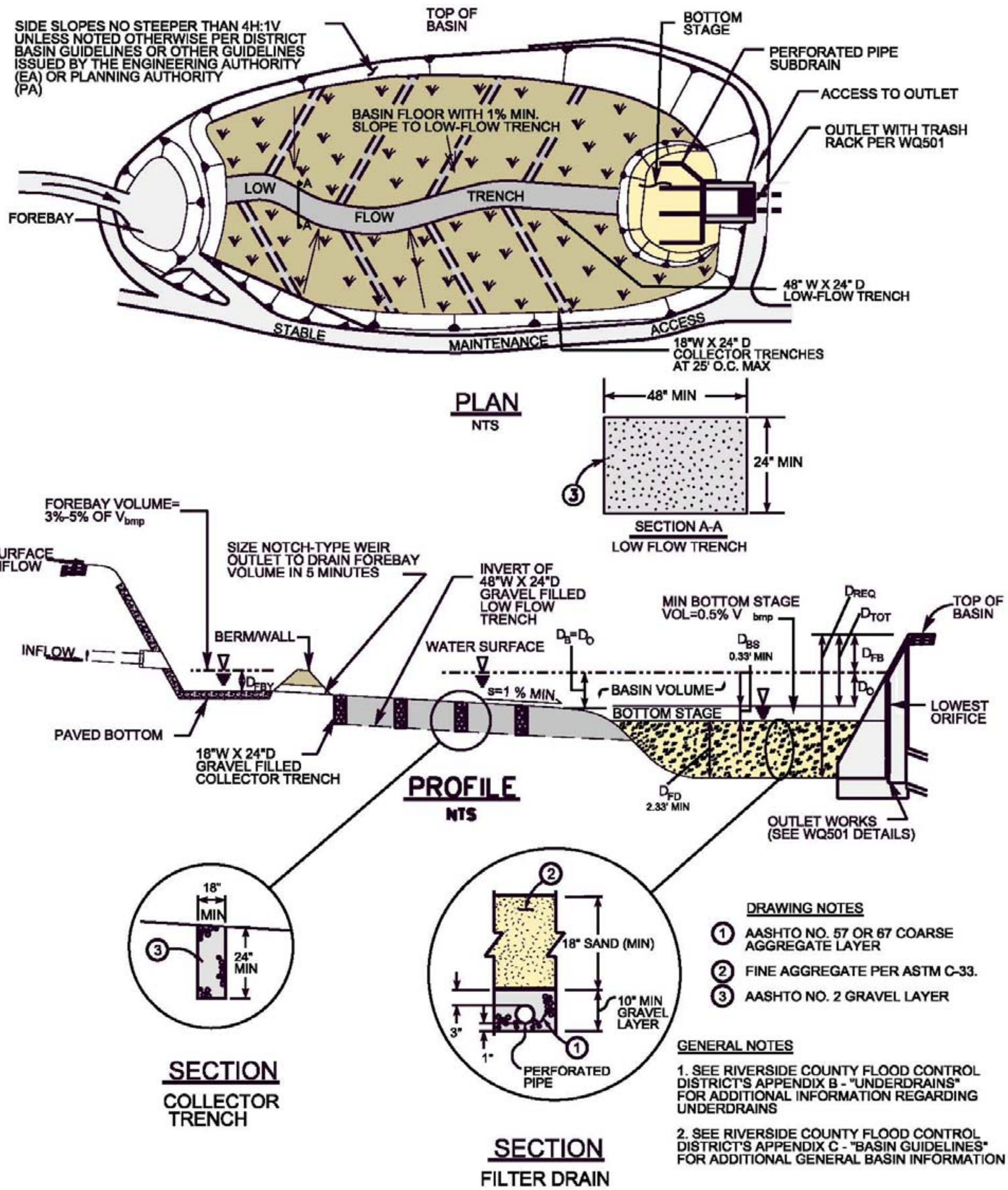
Setbacks

Due to the infiltration characteristics incorporated into the EDB design, the lowest pervious point (beneath the filter drain) of the extended detention facility should be a minimum of 10' above the seasonal high groundwater table. All other setbacks should be in accordance with applicable standards and guidelines issued by the local land use authority.

Basin Guidelines

Additional requirements may be required by the local land use authority (i.e., fencing, maintenance access, etc.).

Figure 1 – Extended Detention Basin



A.4.3 Extended Detention Basin Design Criteria

Design Parameter	Extended Detention Basin
Drawdown time (total)	48 hours
Minimum drawdown time for 50% V_{BMP}	24 hours
Minimum tributary area	five acres
Outlet erosion control	Energy dissipaters to reduce velocities
Forebay volume	3 to 5% of V_{BMP}
Basin Invert Longitudinal Slope (min.)	1%
Basin Invert Transverse (cross) Slope (min)	1%
Low-flow trench width (min.)	48 inches
Low-flow trench depth (min.)	24 inches
Slope of low-flow trench along bottom excavated surface (max.)	1%
Slope of gravel collector trenches along bottom excavated surface (max.)	1 %
Length to width ratio (min.)	1.5:1
Basin depth (min.)	1 foot
Bottom stage volume	0.5 % of V_{BMP}
Bottom stage depth (min)	0.33 feet
Filter drain depth (min)	2.33 feet

Note: The information contained in this BMP Factsheet is intended to be a summary of design considerations and requirements. Information herein may be superseded by guidelines issued by the local land use authority.

Landscaping Requirements

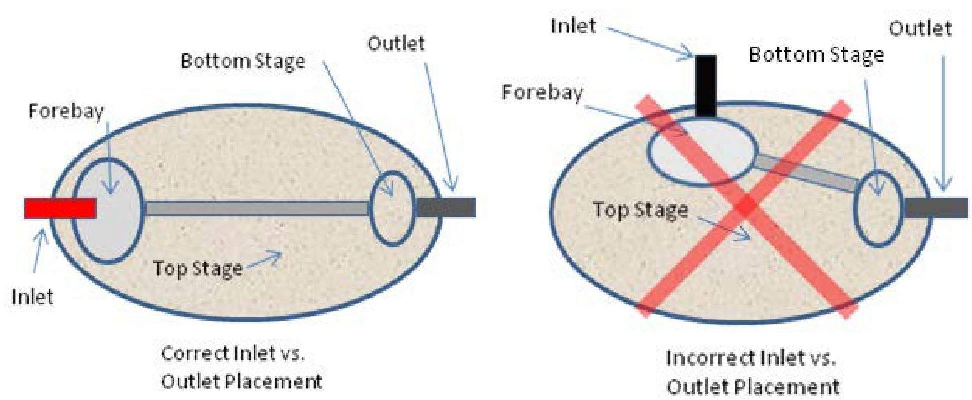
Basin landscaping provides erosion protection, enhances evapotranspiration and infiltration, and improves pollutant removal. The upper stage basin surface, berms and side slopes should be planted with desert appropriate native landscaping. Proper landscape management is also required to ensure that the landscaping does not contribute to water pollution through the use of pesticides, herbicides, or fertilizers. Landscaping should be in accordance with applicable standards and guidelines issued by the local land use authority.

Maintenance Guidelines

Schedule	Inspection and Maintenance Activity
During every scheduled maintenance check (per below), and *as needed* at other times.	• Maintain vegetation as needed. Use of fertilizers, pesticides and herbicides should be strongly avoided to ensure they don't contribute to water pollution. If appropriate native plant selections and other IPM methods are used, such products shouldn't be needed. If such projects are used: o Care should be taken to avoid contact with the low-flow or other trenches, and the media filter in the bottom stage. o Products should be applied in accordance with their labeling, especially in relation to application to water, and in areas subjected to flooding. o Fertilizers should not be applied within 15 days before, after, or during the rainy season. • No ponded water should be present for more than 72 hours to avoid nuisance or vector problems. No algae formation should be visible. Correct problems as needed.
Annually. If possible, schedule these inspections before the beginning of the rain season to allow for any repairs to occur before rains occur.	• Remove debris and litter from the entire basin • Inspect hydraulic and structural facilities. Examine the outlet for clogging, the embankment and spillway integrity, as well as damage to any structural element. • Check for erosion, slumping and overgrowth. Repair as needed. • Inspect sand media at the filter drain to verify it is allowing acceptable infiltration. **Scarify top 3 inches by raking the filter drain's sand surface annually.** • Check the media filter underdrains (via the cleanout) for damage or clogging. Repair as needed. • Remove accumulated sediment and debris from the forebay, and ensure that the notch weir is clear and will allow proper drainage. • Check gravel filled low-flow and collector trenches for sediment buildup and repair as needed.
Every 5 years or sooner (depending on whether observed drain times to empty the basin are less than 48 hours).	• Remove the top 3 inches of sand from the filter drain and backfill with 3 inches of new sand to return the sand layer to its original depth. When scarification or removal of the top 3 inches of sand is no longer effective, remove and replace sand filter layer.
Whenever substantial sediment accumulation has occurred.	• Remove accumulated sediment from the bottom of the basin. Removal should extend to original basin depth.

A.4.4 Extended Detention Basin Design Procedure

1. Find the Design Volume, V_{BMP}.
   a) Enter the Tributary Drainage Area, A_{TRIB}.
   b) Enter the Design Volume, V_{BMP}, determined from **Worksheet 1** of this Handbook.
2. Basin Footprint
   a) Enter the length and width of the EDB. The length should be measured between the inlet to the basin and the outlet structure; and the width should be measured at the widest point of the basin invert. The length to width ratio should be 1.5:1 or longer to prevent short-circuiting and increase the overall effectiveness of the BMP.



   b) Enter the internal basin side slopes. If variable internal side slopes are used, enter the steepest slope that will be used.
   c) Using Figure 1 as a guide, enter the proposed basin depth, D_B, and the freeboard depth, D_{FB} . Based on the information provided, the spreadsheet will calculate the minimum total depth required, D_{REQ}, for this BMP. D_{REQ} is the depth from the bottom of the underdrain layer in the bottom stage (see step 5c), to the top of the freeboard. This calculated minimum required depth can be used to determine if enough elevation difference is available within the design topography to allow for use of this BMP.
   d) Additionally, the basin depth D_B is equal to D_O, which is the depth from the design pond water surface elevation to the lowest orifice in the outlet structure. D_O, is confirmed by the spreadsheet and is used in the basin outlet design described in step 6 below. It should be noted that this lowest orifice is a critical elevation in the design of this BMP. The volume of the basin V_{BASIN} described in Step 3d) is the volume of water above this lowest orifice. This lowest-orifice also represents the dry weather ponded water surface discussed in Step 5c below. Below this elevation there must be a minimum of a 4 inch drop down to the surface of the sand filter in the bottom stage.
3. Basin Design
   a) The Total Basin Depth, D_{TOT} , is calculated automatically, and is the sum of the basin depth D_B plus the freeboard depth D_{FB}.

b) Enter the longitudinal slope of the basin invert. This slope must be at least 1% and is measured along the low-flow trench between the forebay and the bottom stage. Note that the surface of the sand layer in the bottom stage must be level (see Figure 1).

c) Enter the transverse slope of the basin invert. This transverse (cross-sectional) slope must be at least 1% sloped toward the low-flow trench.

d) Enter the volume of the basin, V_{BASIN}. This volume must be the actual volume of water held within the basin as substantiated by modeling or appropriate volumetric calculations, and must be equal to or greater than V_{BMP}. This volume must be held above the lowest orifice in the basin outlet design described in Step 6 below.

4. Forebay Design

All flows must enter the basin through the forebay. The forebay provides a location for the settlement and collection of larger particles, and any other trash or debris. A relatively smooth and level concrete bottom surface should be provided to facilitate mechanical removal of any accumulated sediment, trash, and debris.

Figure 2: Forebay filled with stormwater

a) Enter the forebay volume VFB. This volume must be from 3 to 5% of V_{BMP}.

b) A rock or concrete berm must be constructed to detain water before it drains into the basin. The top of the berm should be set no higher than the invert of the inlet conveyance. Enter the forebay depth, D_{FBY}.

c) The spreadsheet will calculate the minimum surface area of the forebay, A_{FB}, based on the provided forebay volume and depth. Ensure that the plans provide for a forebay area at least this large.

d) Although the forebay will be well submerged in the design event, a full height rectangular notch-type weir should be constructed through the berm to prevent permanent ponding in the forebay, and allow water to slowly and fully drain to the main body of the basin. This notch should be offset from the inflow streamline to prevent low-flows from short circuiting. Enter the width (W) of this rectangular notch weir. The width should not be less than 1 $^{1/2}$ inches to prevent clogging. Additionally, immediately outside the notch construct a minimum one foot by one foot gravel pad to prevent vegetative growth within the basin invert from blocking the notch.

5. Dry Weather and Low-Flow Management

The basin should have both a low-flow gravel trench and a network of gravel collector trenches across the invert of the basin, as well as a bottom stage sand filter to treat low-flows and dry weather flows (see Figure 1).

a) Low-Flow Trench: The low-flow gravel trench conveys flow from the forebay to the bottom stage, while allowing for maximum incidental infiltration and volume loss. The trench should be a minimum of 48 inches wide by 24 inches deep. This trench should be unlined and backfilled with AASHTO No. 2 gravel (or similar) to the finished surface of the basin invert, and should not use underdrains. The bottom excavated surface of the low-flow trench should be 1% or flatter to promote infiltration.

Figure 3: Gravel filled low-flow trench

b) Collector Trenches: Gravel collector trenches beneath the top stage should be arranged as illustrated in Figure 1 of page 53 above, with minimal slope (1% maximum) along their bottom excavated surface to promote infiltration, and must extend from the low-flow trench to the toe of the basin side slopes. They should be a minimum of 18 inches wide by 24 inches deep, unlined and backfilled with AASHTO No. 2 gravel (or similar) to the finished basin invert surface. The gravel collector trenches should not use underdrains and should be constructed with a maximum spacing of 25 feet, center to center (see Figure 1 on page 53 above).

c) Bottom Stage: A depressed sand filter drain area, referred to as the bottom stage, must be constructed adjacent to the outlet structure to treat any dry weather flows. To ensure that dry weather flows are treated through the sand filter and not discharged through the orifice plate, the top surface of the sand filter must be depressed at least four inches below the lowest orifice in the outlet structure. This depressed area will create a micro pool of water that is then filtered down through the sand filter and out through underdrains. Based on the minimum dimensions described below, the minimum depth of excavation below the lowest orifice in the outlet structure is 2.33 feet.

i. Enter the depth of the bottom stage, D_{BS}. As mentioned above, this depth must be at least four inches, and extend down below the lowest orifice in the outlet structure.

ii. Enter the area of the bottom stage, A_{BS}.

iii. Based on the D_{BS} and A_{BS} entered, the spreadsheet will calculate V_{BS}. This volume is the volume of ponded water that will be held below the lowest orifice in the outlet structure, and above the surface of the sand filter. This volume must be at least 0.5% of V_{BMP}.

iv. Enter the thickness of the ASTM C-33 sand layer that will be provided, D_S. A minimum thickness of 18 inches is required.

v. Below the sand layer, a minimum 10 inch thick layer of gravel should be installed with underdrains to drain the water that has been treated through the sand filter.

The underdrains should connect into the outlet structure. See Appendix C for underdrain construction. Enter the diameter of the underdrain pipe (minimum 6" dia.), and the spacing of the underdrains. The maximum spacing of the underdrains is 20 feet on center, however, where the area of the bottom stage is particularly small (less than 500 square feet), the underdrain pipes should be placed at no more than a 10 foot separation on center.

6. Basin Outlet Design

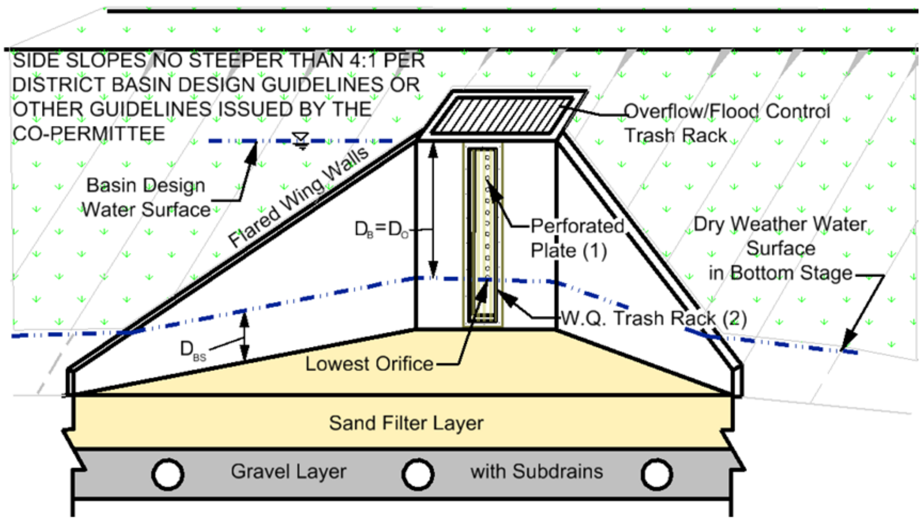


Figure 4: Basin Outlet Structure with Bottom Stage Shown

Outlet structures for publicly maintained basins shall conform to District Standard Drawings WQ501 unless approved in advance by the local land use authority. This standardization is to provide for efficient maintenance. The basin outlet should be sized to release the design volume, V_{BMP}, within a 48-hour period but 50% of V_{BMP} within 24 hours. This is an iterative design process where an appropriate control orifice can be selected using the following steps:

a) Develop a Stage vs. Discharge Curve for the Outlet Structure

Estimate the orifice size and outlet plate configuration (number per row, etc.). Based on D_O provided in the Basin Footprint Section, the spreadsheet will automatically generate the stage vs. discharge relationship for this outlet:

$$Q = C*A*[2*g*(H-H_o)]^{0.5}$$

Where:

Q = discharge (ft^3/s)
C = orifice coefficient
A = area of the orifice (ft)
g = gravitational constant (32.2 ft2/s)
H = water surface elevation (ft)
H_o = orifice elevation (ft)

The lowest orifice shall be located with its centerline at the top of the bottom stage, at least four inches above the surface of the sand filter drain. To help avoid clogging, the minimum orifice diameter is limited to 3/8 inch. Since the 1/4 inch thickness of the orifice plate will be less than the orifice diameter, a value for C of 0.66 may be used. If another value for C is used, justification may be required.

b) Develop a Discharge/Volume vs. Stage Table for the Basin

Based on the shape and size of the basin, develop a relationship between the stage and the volume of water in the basin. Since the orifice spacing is four inches on center for the standard orifice plate, the stage intervals must also be four inches. Enter the basin volume at each interval starting at the centerline of the lowest orifice.

c) Route the Design Volume through the Basin

The spreadsheet assumes that the design volume (V_{BMP}) enters the basin instantaneously and as such, no inflow/outflow hydrograph is necessary. The drawdown time for each stage becomes:

$$\Delta t = V_i/Q$$

Where:

Δt = drawdown time for each stage
V_i = the volume at each stage
Q = the flow rate corresponding to the headwater elevation at each stage.

The spreadsheet automatically determines the drawdown time from the sum of the Δt values for each stage. If the orifice size and plate configuration estimate meets the hydraulic retention time requirements (50% of the volume empties in not less than 24 hours, 100% of the volume empties in no more than 48 hours), the outlet is correctly sized. If these requirements are not met, select a new orifice size or configuration and repeat the process starting at Step 6a.

7. Outlet Protection

   To prevent the orifices from clogging, trash racks are required where perforated vertical outlet control plates are used. This allows for easier access to outlet orifices for inspection and cleaning. Trash racks shall be sized to prevent clogging of the primary water quality outlet without restricting the hydraulic capacity of the outlet control orifices. The orifice plate should be protected with a trash rack conforming to Standard Drawing WQ501 (at end of this section) with at least six square feet of open surface area or 25 times the total orifice area, whichever is greater. The rack shall be adequately secured to prevent it from being removed or opened when maintenance is not occurring.

Trash Rack with Screen

Overflow Structure Similar to Standard Drawing Number WQ 501
(Photo courtesy of Colorado Association of Stormwater Floodplain Managers)

8. Overflow Outlet

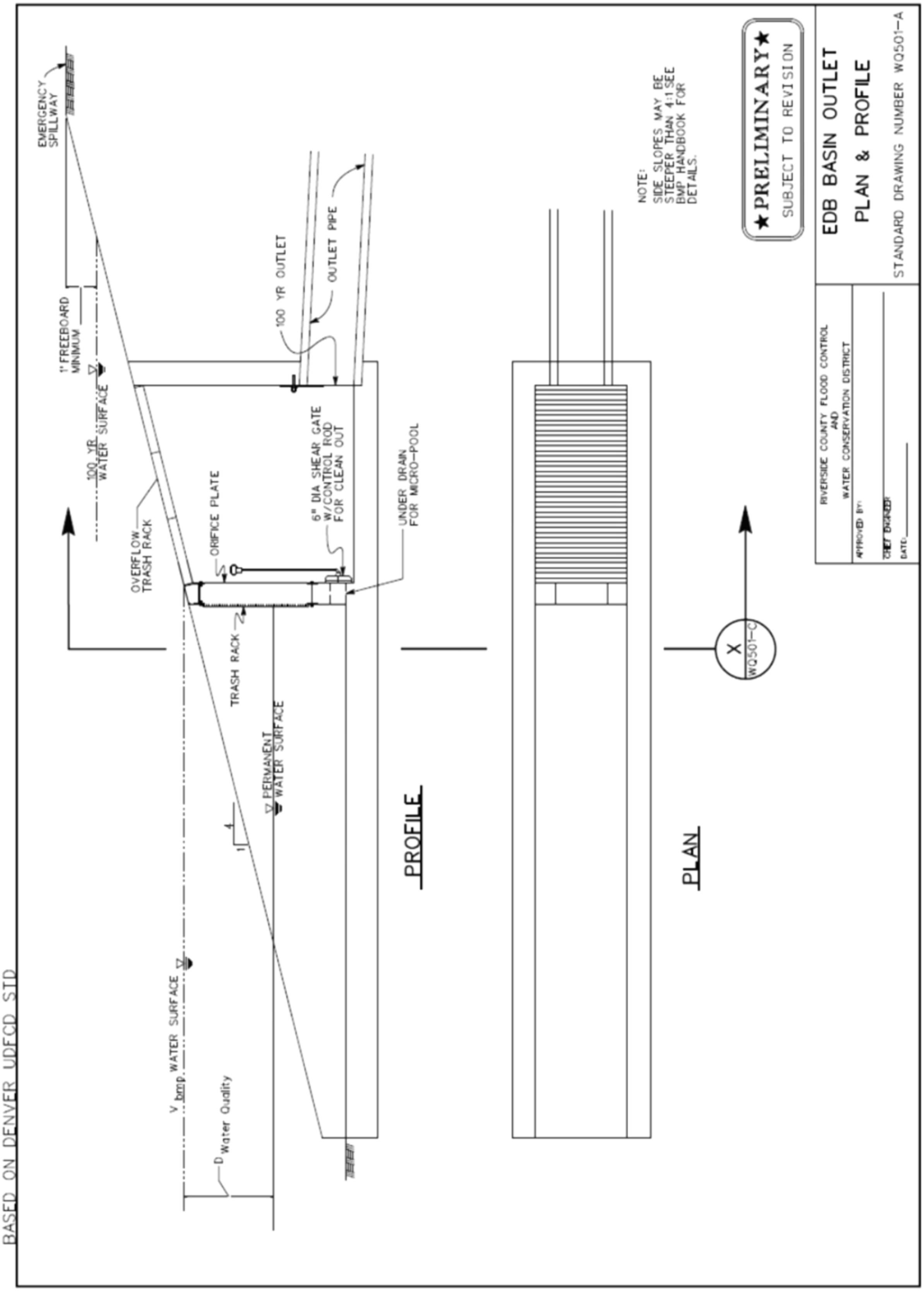
   Overflow outlets for publicly maintained basins shall conform to Standard Drawing WQ501 (at end of this section) unless approved in advance by the local land use authority.

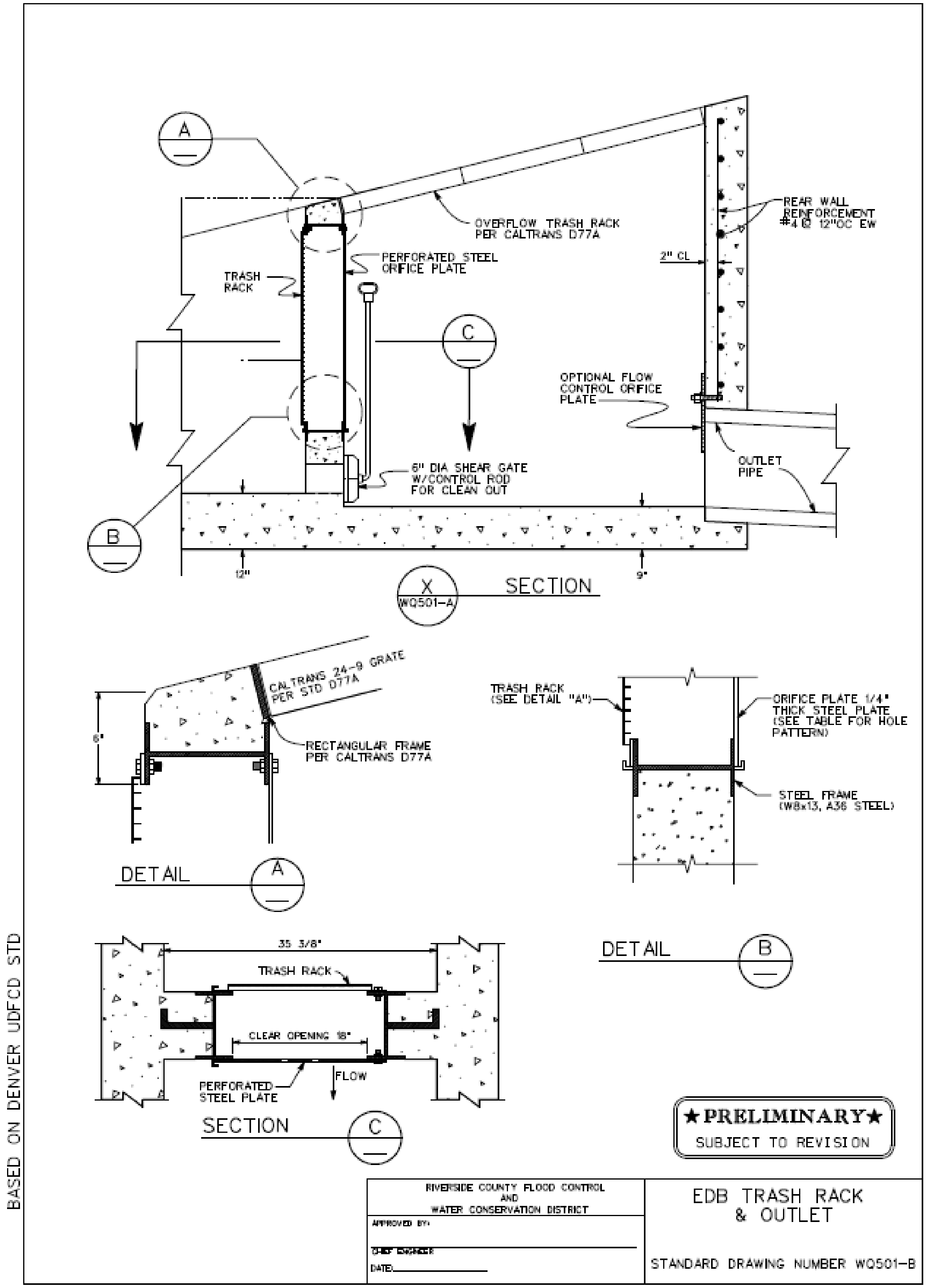
9. Embankment

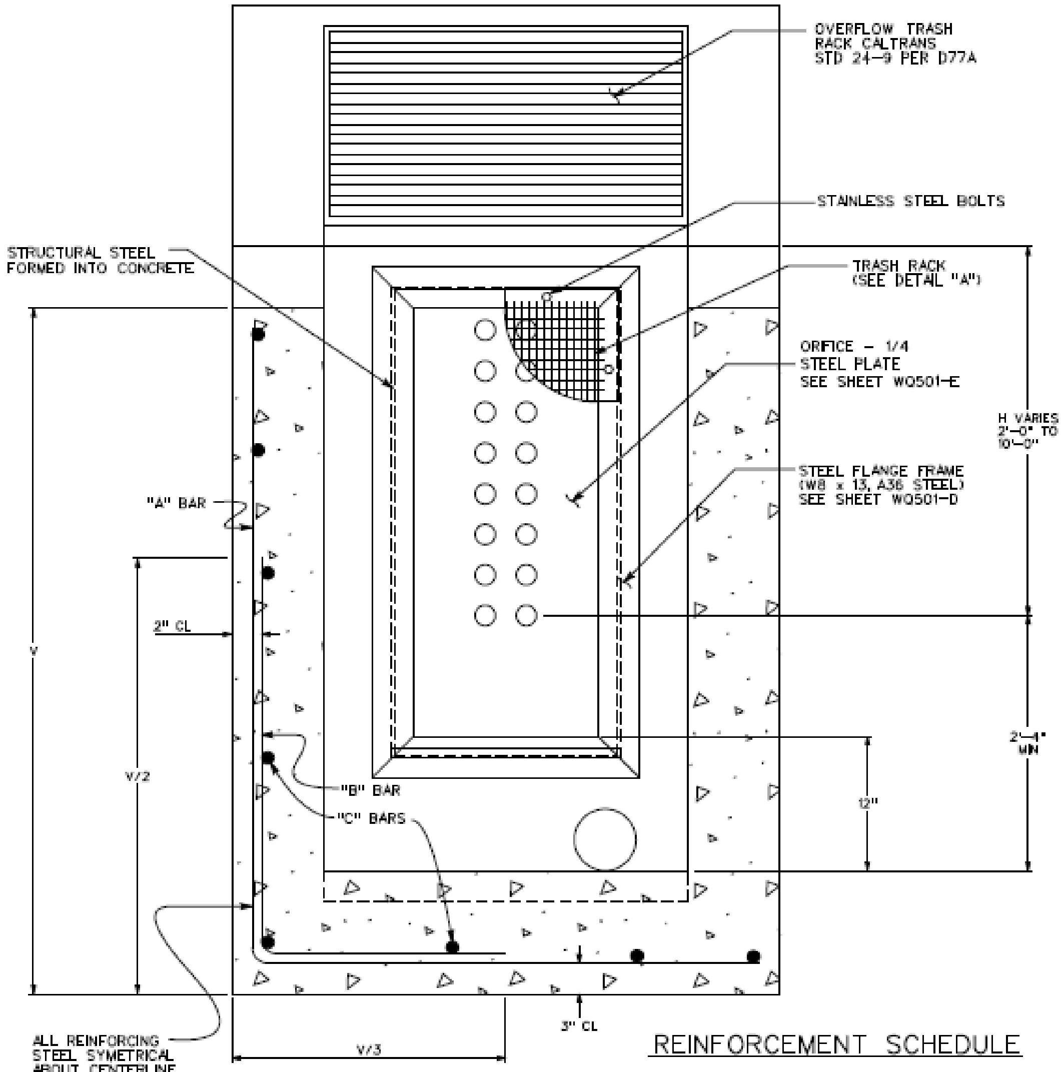
   Embankments shall be designed in accordance with applicable standards and guidelines issued by the local land use authority. Where applicable, embankment designs must additionally conform to the requirements of the State of California Division of Safety of Dams.

10. Spillway and Overflow Structures

    Spillway and overflow structures should be designed in accordance with applicable standards and guidelines issued by the local land use authority.


EMERGENCY SPILLWAY
1' FREEBOARD MINIMUM
100 YR WATER SURFACE
100 YR OUTLET
OUTLET PIPE
NOTE:
SIDE SLOPES MAY BE STEEPER THAN 4:1 SEE BMP HANDBOOK FOR DETAILS.
★PRELIMINARY★
SUBJECT TO REVISION
EDB BASIN OUTLET
PLAN & PROFILE
STANDARD DRAWING NUMBER WQ501-A
RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT
APPROVED BY:
CHIEF ENGINEER
DATED:
OVERFLOW TRASH RACK
ORIFICE PLATE
6" DIA SHEAR GATE W/CONTROL ROD FOR CLEAN OUT
UNDER DRAIN FOR MICRO-POOL
TRASH RACK
PERMANENT WATER SURFACE
X
WQ501-C
4
1
PROFILE
PLAN
V bmp WATER SURFACE
D Water Quality
BASED ON DENVER UDFCD STD


OVERFLOW TRASH RACK PER CALTRANS D77A
REAR WALL REINFORCEMENT #4 @ 12"OC EW
PERFORATED STEEL ORIFICE PLATE
TRASH RACK
2" CL
OPTIONAL FLOW CONTROL ORIFICE PLATE
OUTLET PIPE
6" DIA SHEAR GATE W/CONTROL ROD FOR CLEAN OUT
12"
9"
SECTION
WQ501-A
CALTRANS 24-9 GRATE PER STD D77A
RECTANGULAR FRAME PER CALTRANS D77A
6"
DETAIL A
TRASH RACK (SEE DETAIL "A")
ORIFICE PLATE 1/4" THICK STEEL PLATE (SEE TABLE FOR HOLE PATTERN)
STEEL FRAME (W8x13, A36 STEEL)
DETAIL B
35 3/8"
TRASH RACK
CLEAR OPENING 18"
PERFORATED STEEL PLATE
FLOW
SECTION C
★PRELIMINARY★
SUBJECT TO REVISION
BASED ON DENVER UDFCD STD
RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT
APPROVED BY:
CHIEF ENGINEER
DATE
EDB TRASH RACK & OUTLET
STANDARD DRAWING NUMBER WQ501-B



REINFORCEMENT SCHEDULE

MAX V	T WALL	A&B BARS	C BARS
6'	9"	#4 @ 18"	#4 @ 18"
10'	10"	#4 @ 12"	#4 @ 18"
12'	10"	#5 @ 12"	#4 @ 18"
REAR WALL REINFORCEMENT #4 @ 12" OC EW SEE DWG WQ501-B			

ELEVATION (X / WQ501-A)

★PRELIMINARY★
SUBJECT TO REVISION

BASED ON DENVER UDFCD STD

RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT

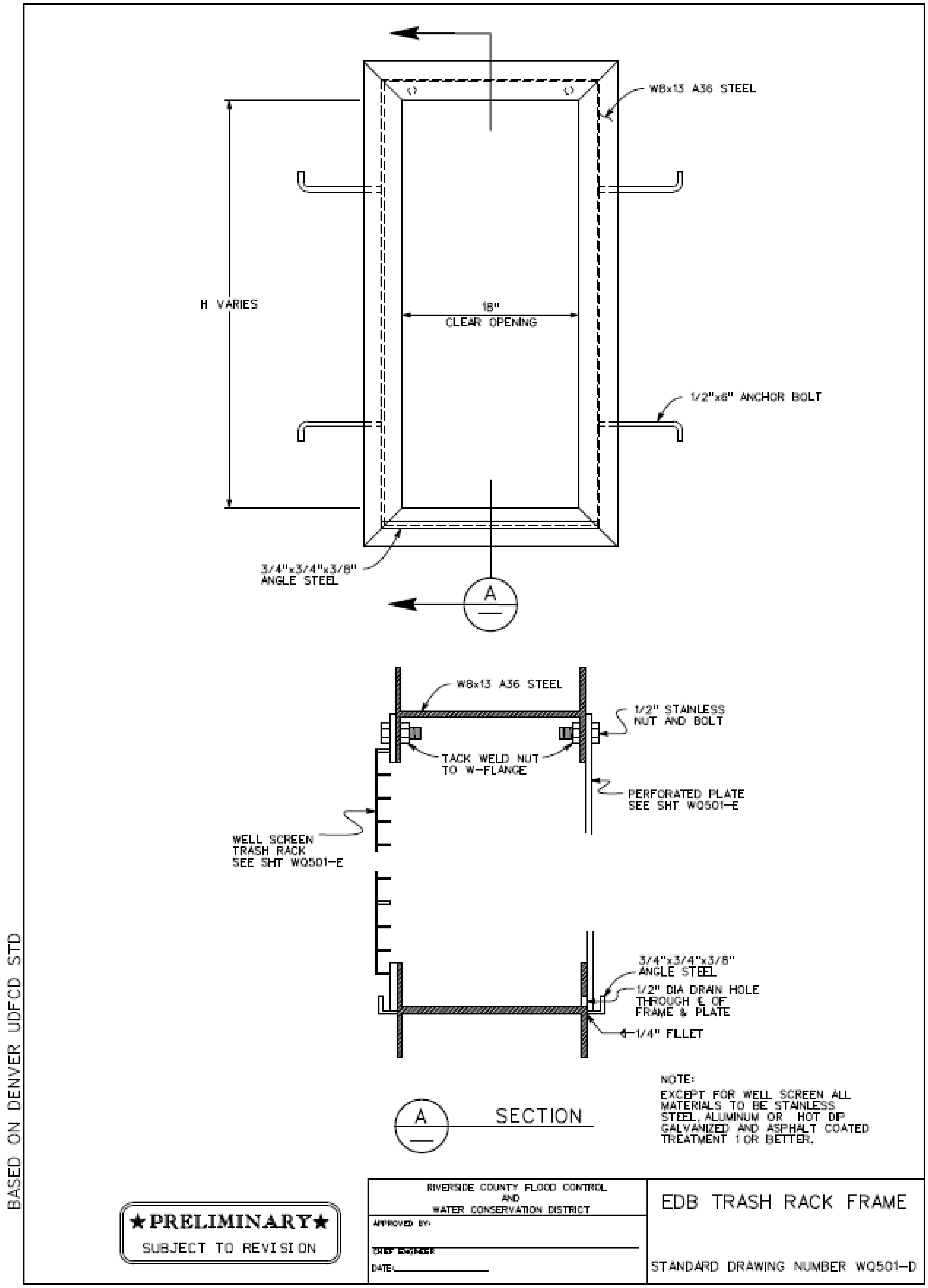
APPROVED BY:

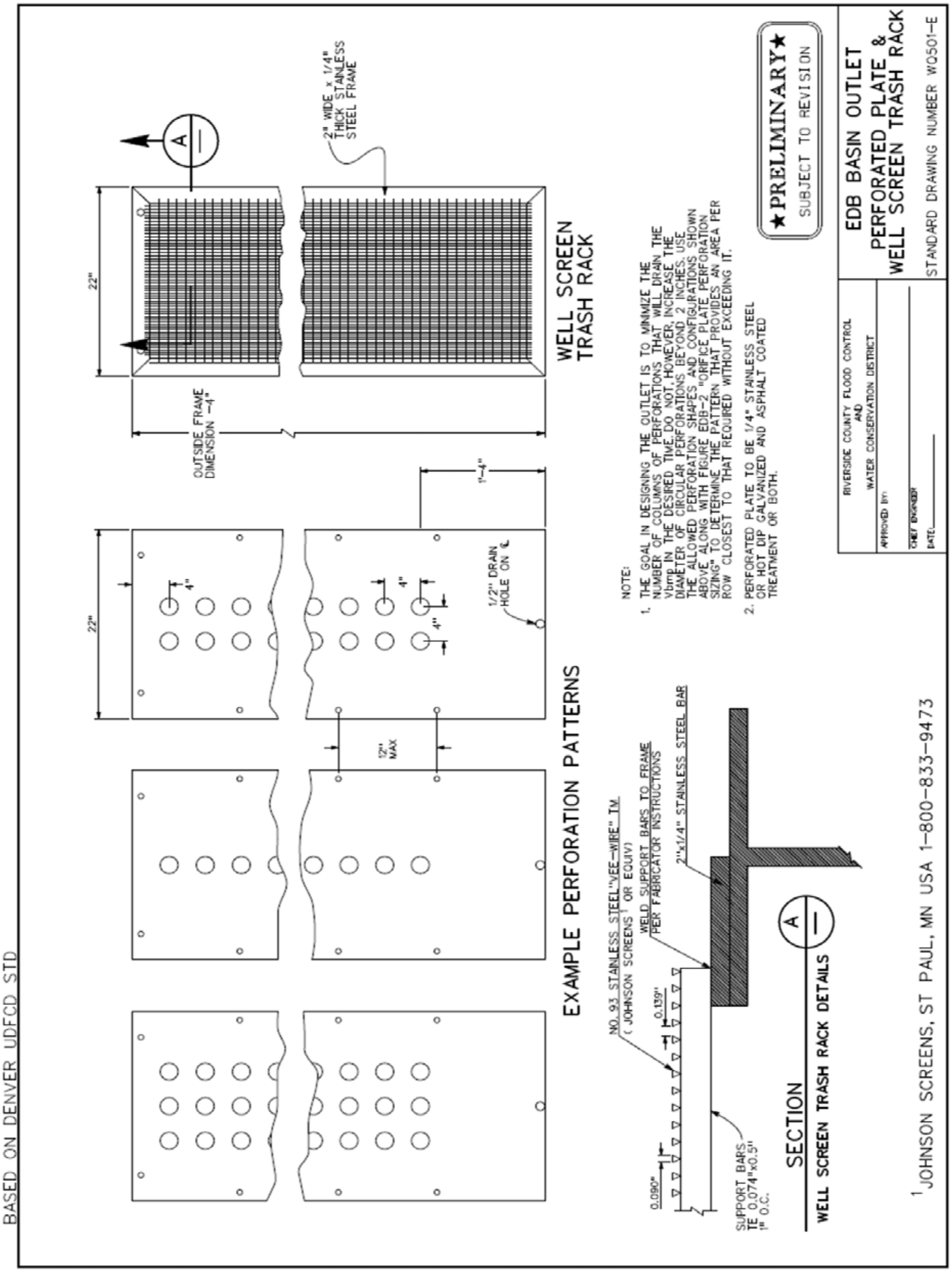
CHIEF ENGINEER

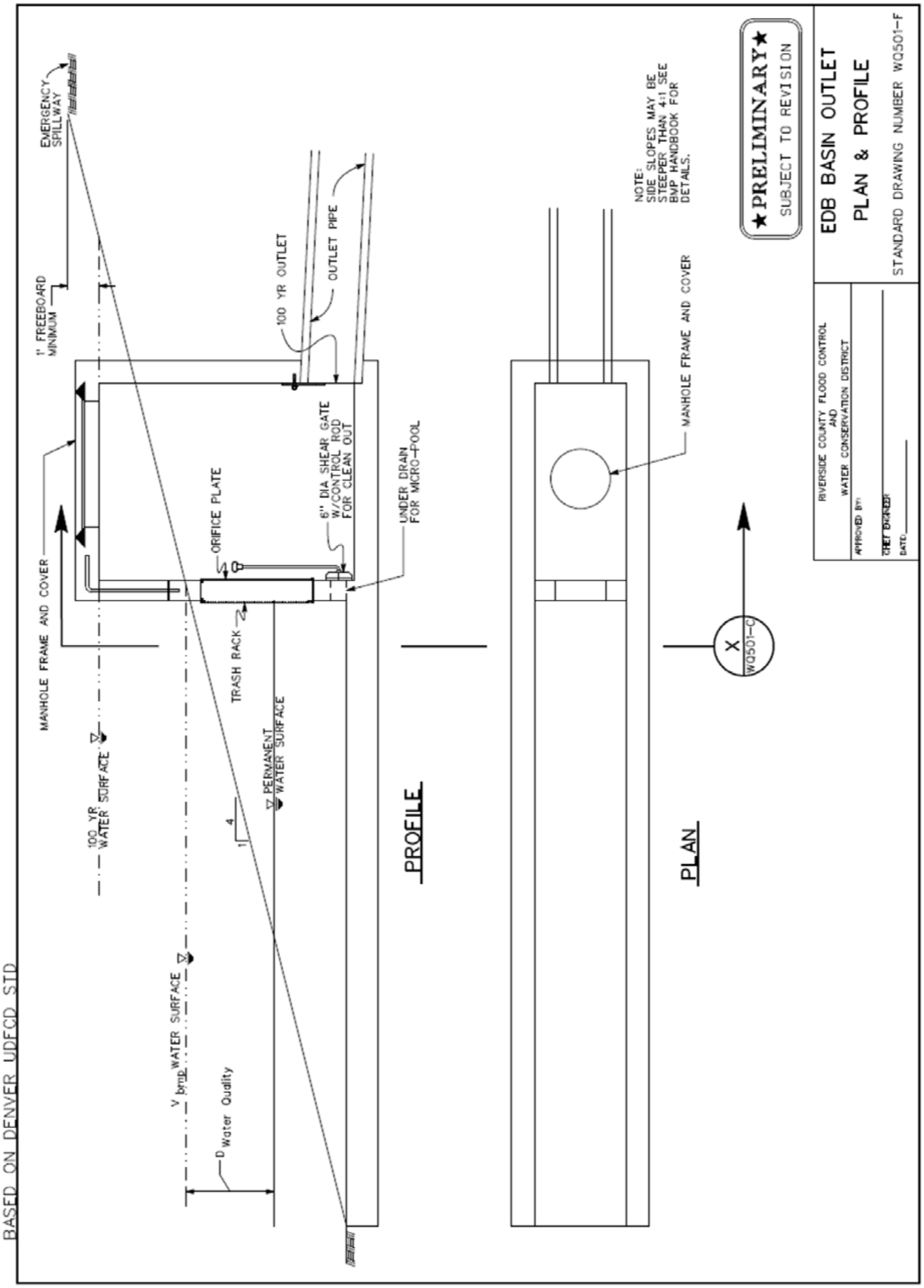
DATE:

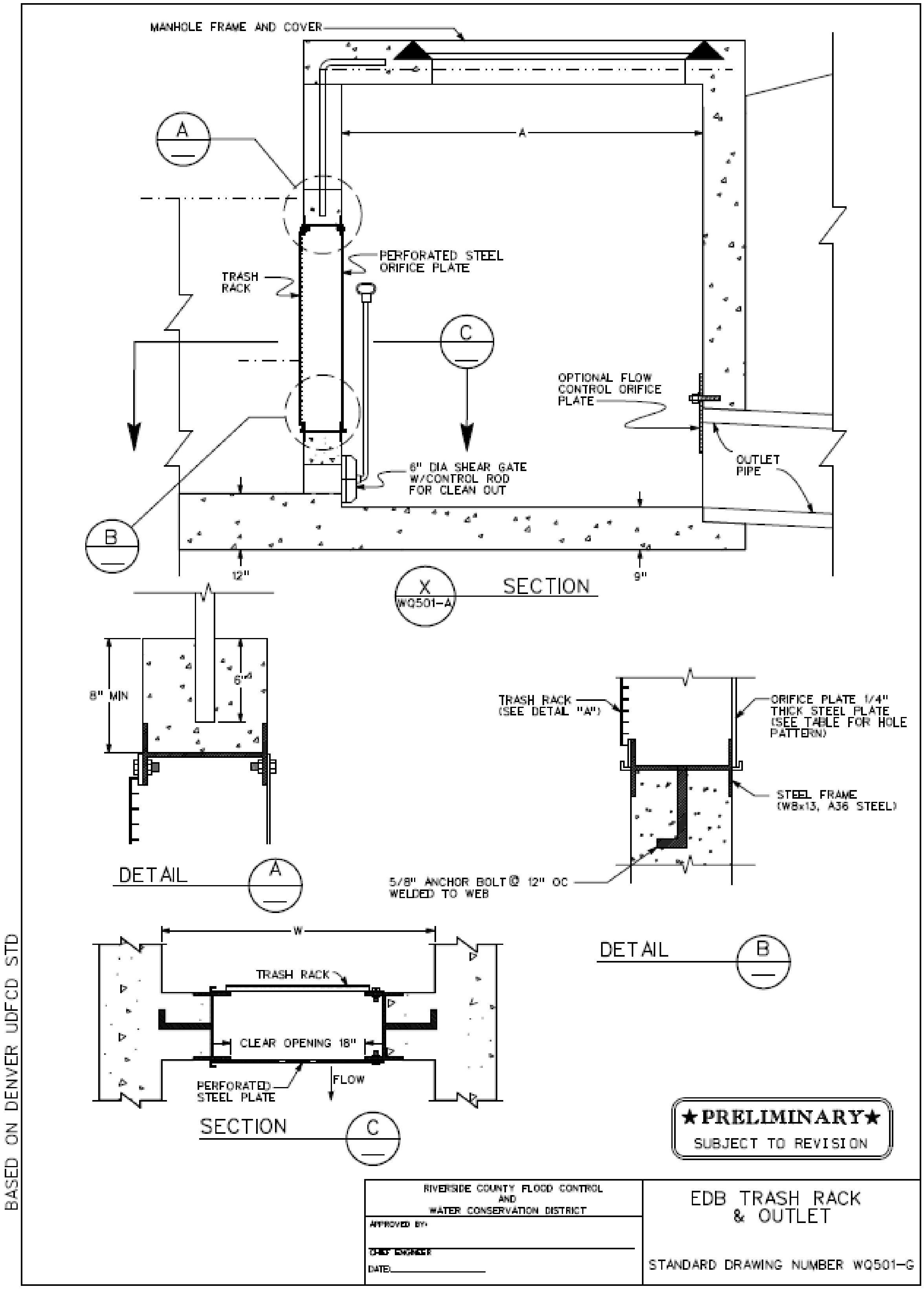
EDB BASIN OUTLET

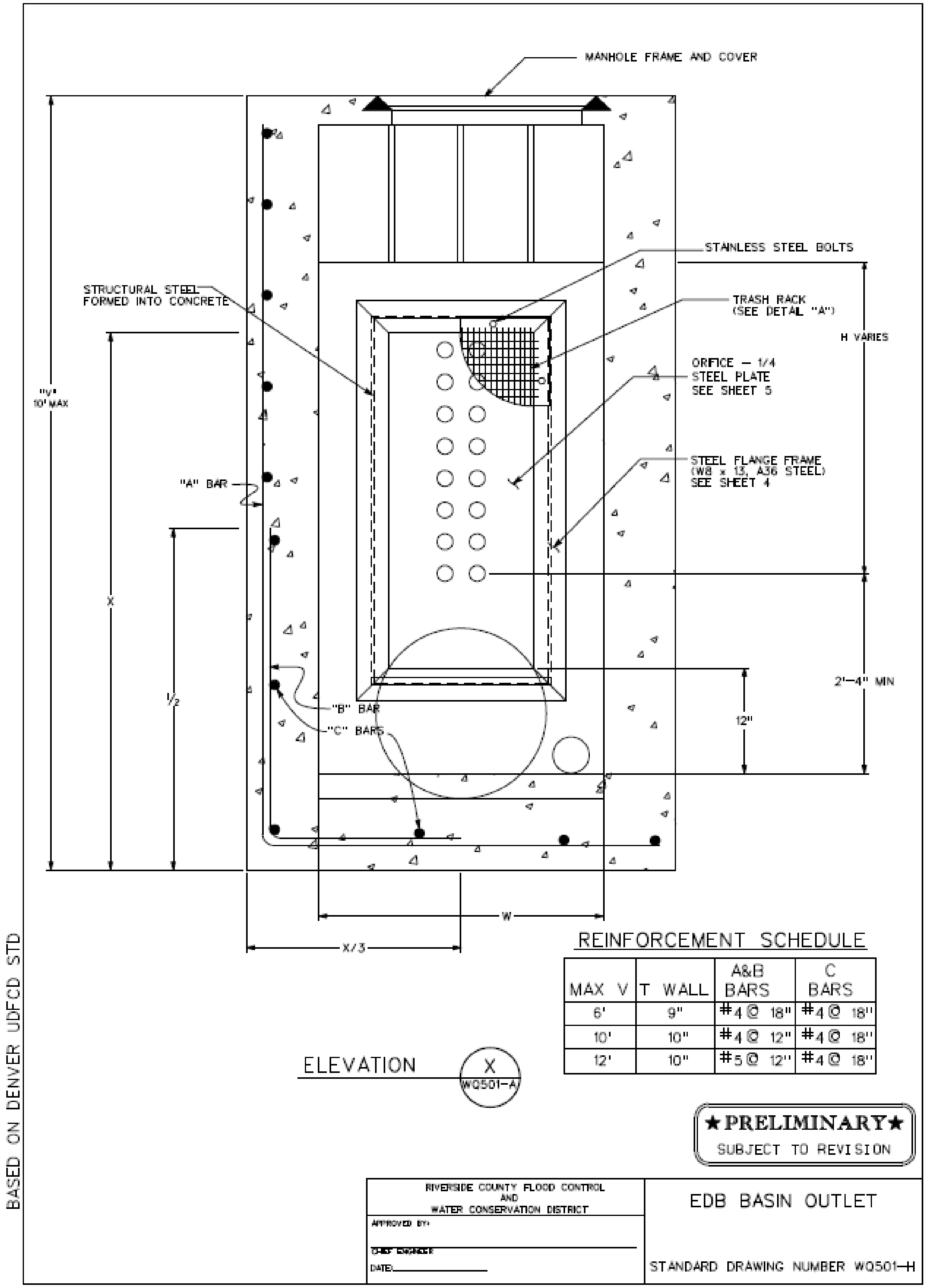
STANDARD DRAWING NUMBER WQ501-C


W8x13 A36 STEEL
H VARIES
18"
CLEAR OPENING
1/2"x6" ANCHOR BOLT
3/4"x3/4"x3/8"
ANGLE STEEL
A
W8x13 A36 STEEL
1/2" STAINLESS
NUT AND BOLT
TACK WELD NUT
TO W-FLANGE
PERFORATED PLATE
SEE SHT WQ501-E
WELL SCREEN
TRASH RACK
SEE SHT WQ501-E
3/4"x3/4"x3/8"
ANGLE STEEL
1/2" DIA DRAIN HOLE
THROUGH ℄ OF
FRAME & PLATE
1/4" FILLET
NOTE:
EXCEPT FOR WELL SCREEN ALL MATERIALS TO BE STAINLESS STEEL, ALUMINUM OR HOT DIP GALVANIZED AND ASPHALT COATED TREATMENT 1 OR BETTER.
A SECTION
BASED ON DENVER UDFCD STD
★PRELIMINARY★
SUBJECT TO REVISION
RIVERSIDE COUNTY FLOOD CONTROL
AND
WATER CONSERVATION DISTRICT
APPROVED BY:
CHIEF ENGINEER
DATE:
EDB TRASH RACK FRAME
STANDARD DRAWING NUMBER WQ501-D


BASED ON DENVER UDFCD STD
22"
4"
12" MAX
1'-4"
1/2" DRAIN HOLE ON ℄
EXAMPLE PERFORATION PATTERNS
OUTSIDE FRAME DIMENSION -4"
A
2" WIDE x 1/4" THICK STAINLESS STEEL FRAME
WELL SCREEN TRASH RACK
NO. 93 STAINLESS STEEL"VEE-WIRE" TM (JOHNSON SCREENS 1 OR EQUIV)
WELD SUPPORT BARS TO FRAME PER FABRICATOR INSTRUCTIONS
0.090"
0.139"
2"x1/4" STAINLESS STEEL BAR
SUPPORT BARS TE 0.074"x0.5" 1" O.C.
SECTION
WELL SCREEN TRASH RACK DETAILS
NOTE:
1. THE GOAL IN DESIGNING THE OUTLET IS TO MINIMIZE THE NUMBER OF COLUMNS OF PERFORATIONS THAT WILL DRAIN THE Vbmp IN THE DESIRED TIME. DO NOT, HOWEVER, INCREASE THE DIAMETER OF CIRCULAR PERFORATIONS BEYOND 2 INCHES. USE THE ALLOWED PERFORATION SHAPES AND CONFIGURATIONS SHOWN ABOVE ALONG WITH FIGURE EDB-2 "ORIFICE PLATE PERFORATION SIZING" TO DETERMINE THE PATTERN THAT PROVIDES AN AREA PER ROW CLOSEST TO THAT REQUIRED WITHOUT EXCEEDING IT.
2. PERFORATED PLATE TO BE 1/4" STAINLESS STEEL OR HOT DIP GALVANIZED AND ASPHALT COATED TREATMENT OR BOTH.
★PRELIMINARY★
SUBJECT TO REVISION
RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT
APPROVED BY:
CHIEF ENGINEER
DATE:
EDB BASIN OUTLET
PERFORATED PLATE &
WELL SCREEN TRASH RACK
STANDARD DRAWING NUMBER WQ501-E
1 JOHNSON SCREENS, ST PAUL, MN USA 1-800-833-9473


EMERGENCY SPILLWAY
1' FREEBOARD MINIMUM
100 YR OUTLET
OUTLET PIPE
MANHOLE FRAME AND COVER
ORIFICE PLATE
6" DIA SHEAR GATE W/CONTROL ROD FOR CLEAN OUT
UNDER DRAIN FOR MICRO-POOL
TRASH RACK
100 YR WATER SURFACE
PERMANENT WATER SURFACE
V bmp WATER SURFACE
D Water Quality
PROFILE
PLAN
X WQ501-C
NOTE: SIDE SLOPES MAY BE STEEPER THAN 4:1 SEE BMP HANDBOOK FOR DETAILS.
★PRELIMINARY★ SUBJECT TO REVISION
EDB BASIN OUTLET
PLAN & PROFILE
STANDARD DRAWING NUMBER WQ501-F
RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT
APPROVED BY:
CHIEF ENGINEER
DATED:
BASED ON DENVER UDFCD STD


MANHOLE FRAME AND COVER
A
TRASH RACK
PERFORATED STEEL ORIFICE PLATE
C
OPTIONAL FLOW CONTROL ORIFICE PLATE
OUTLET PIPE
6" DIA SHEAR GATE W/CONTROL ROD FOR CLEAN OUT
B
12"
9"
X WQ501-A SECTION
8" MIN
6"
DETAIL A
TRASH RACK (SEE DETAIL "A")
ORIFICE PLATE 1/4" THICK STEEL PLATE (SEE TABLE FOR HOLE PATTERN)
STEEL FRAME (W8x13, A36 STEEL)
5/8" ANCHOR BOLT @ 12" OC WELDED TO WEB
DETAIL B
W
TRASH RACK
CLEAR OPENING 18"
PERFORATED STEEL PLATE
FLOW
SECTION C
★PRELIMINARY★
SUBJECT TO REVISION
BASED ON DENVER UDFCD STD
RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT
APPROVED BY:
CHIEF ENGINEER
DATE
EDB TRASH RACK & OUTLET
STANDARD DRAWING NUMBER WQ501-G



REINFORCEMENT SCHEDULE

MAX V	T WALL	A&B BARS	C BARS
6'	9"	#4 @ 18"	#4 @ 18"
10'	10"	#4 @ 12"	#4 @ 18"
12'	10"	#5 @ 12"	#4 @ 18"

Worksheet 6 – This worksheet is available on the District's Municipal Stormwater Management Program page for developers at: www.rcflood.org/NPDES/developers.aspx

Extended Detention Basin Design Procedure (Revised 06-2014)	BMP Subarea No.		Legend:	Required Entries / Calculated Cells
Company Name:			**Date:**	
Designed by:			**County/City Case No.:**	

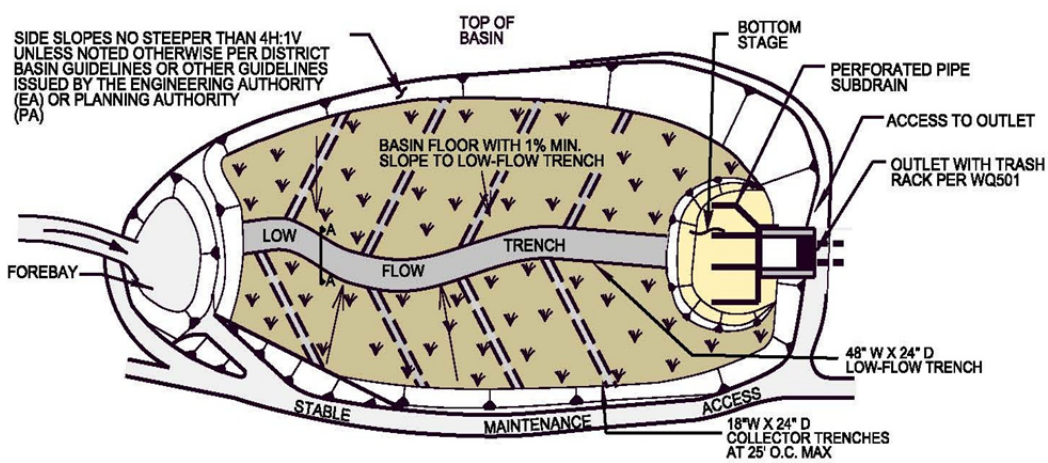
Design Volume

Tributary Area (BMP Subarea)	A_{TRIB} =		acres
Enter V_{BMP}, determined from Section 4.3 of this Handbook	V_{BMP}=		ft^3

Basin Footprint

Overall Geometry

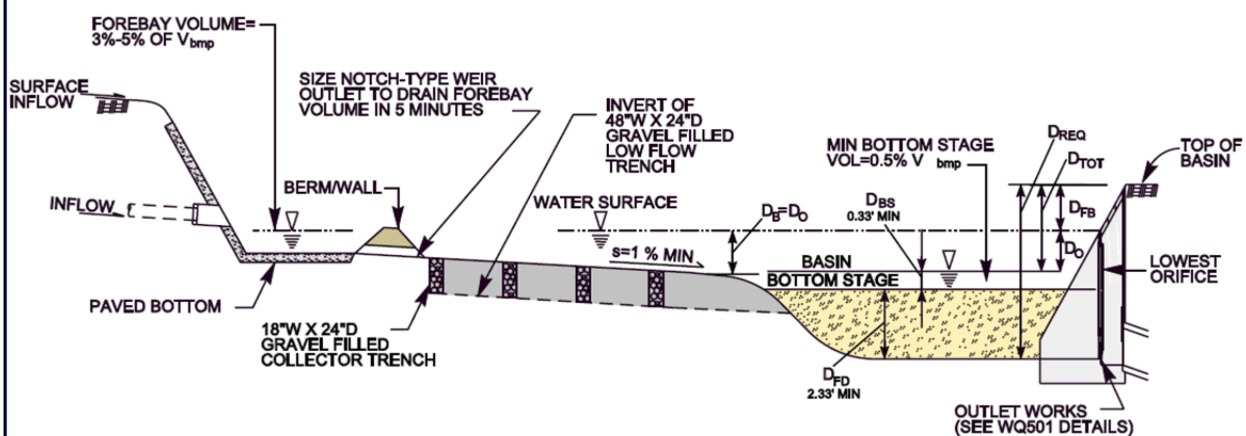
Length at Basin Bottom Surface	Length =		ft
Width at Basin Bottom Surface	Width =		ft
	Meets 1.5 : 1 requirement?		
Side Slopes per "Basin Guidelines", Sect.	z =		:1
Proposed Basin Depth (with no freeboard)	D_B =		ft
Depth of freeboard (if used)	D_{FB} =		ft
Minimum Required Allowance for Total Depth (including proposed basin depth, freeboard, minimum depth of bottom stage (D_{BS}=0.33') and minimum filter depth (D_{FD}=2.33'))	D_{REQ} =		ft
Depth from design water surface elevation to lowest orifice	D_O =		ft



Basin Design

Basin Design

Proposed Total Basin Depth (proposed depth plus freeboard)	D_{TOT} =		ft
Basin Invert Longitudinal Slope	Slope =		%
Basin Invert Transverse Slope (1% min)	Slope =		%
Basin Volume	V_{Basin} =		ft^3



Forebay Design

Forebay Volume (3 - 5% V_{BMP})	V_{FB} =		ft^3
Forebay Depth (height of berm)	D_{FBY} =		ft
Minimum Forebay Surface Area	A_{FB} =		ft^2
Rectangular weir (notch)	W =		in

Dry Weather and Low-Flow Management

Low-Flow Trench (see graphic below)

Depth (24 inches minimum, gravel filled) — Depth = ______ inches

Width (48 inches minimum) — Width = ______ inches

Trench Invert Longitudinal Slope — Slope = ______ %

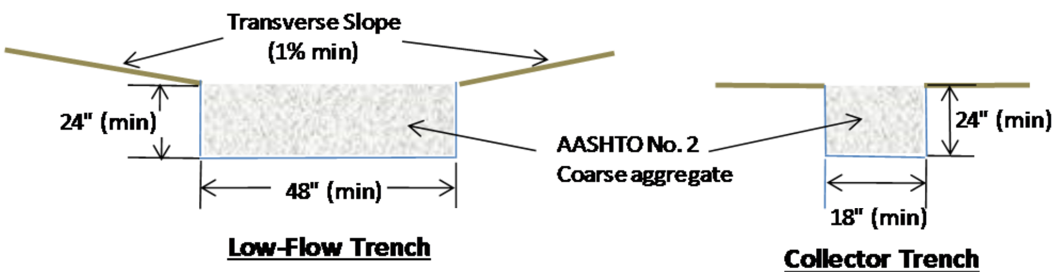
Collector Trenches (see graphic below)

Depth (24 inches minimum) — Depth = ______ inches

Width (18 inches minimum) — Width = ______ inches

Trench Invert Longitudinal Slope — Slope = ______ %

Spacing (25 feet on center maximum) — S = ______ feet



Bottom Stage (Sand Filter) Design

Depth of the Bottom Stage (4" minimum ponding) — D_{BS} = ______ in

Surface Area of Bottom Stage — A_{BS} = ______ ft^2

Dry Weather Ponded Volume (above sand layer) — V_{BS} = ______ ft^3

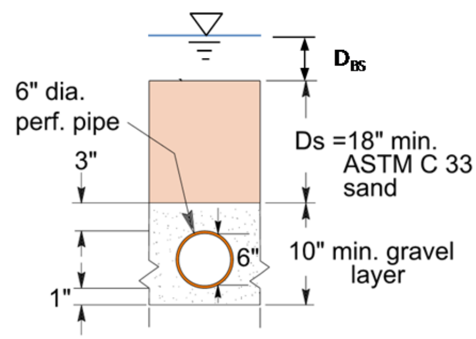
Is V_{BS} no less than 0.5% V_{BMP}? Enter Depth and Surface Area

Depth of ASTM-C33 sand (18 inch minimum)

Ds = ______ inches

Diameter of Subdrains — ϕ = ______ in

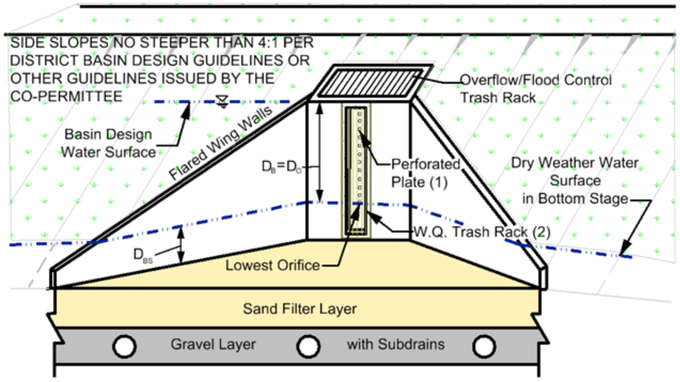
Subdrain Spacing — s = ______ ft. on center



Basin Outlet Design

Outlet Design

Assume an orifice area. Based on the information provided above, the spreadsheet provides discharge vs. stage data. Enter the volume vs. stage data for each interval. This information is used to route the volume through the basin. The size of the orifice is acceptable when the data shows that less than 50% of V_{BMP} has drained in 24 hours, and that 100% drawdown occurs within 48 hours.



All values on this worksheet must be filled out to use this calculator

Flow Rate, Q (cfs)

$$Q=CA[2g(H-H_o)]^{0.5}$$

Discharge Coefficient,

Default, C = 0.66

Other, C =

Orifice Area (ft²)

Orifice Diameter, d; number of orifices per row, n; and number of orifice rows, N (from the bottom up).

d = inches

n = per row

N = rows

Aeff = 0.000 ft² per row

or

Aeff = 0.000 in² per row

From outflow hydrograph, the time where 50% of V_{BMP} has drained from the basin (24 hour minimum):

Time (50%) = Error * hrs

From outflow hydrograph, the time where 100% V_{BMP} has drained from the basin(within 48 hours):

Time (100 %) = Error * hrs

Headwater Elev. / Stage (ft)	Discharge (cfs)	Volume (acre-ft)	Δt (hrs.)
0	0.0000	0.0000	
0.33			
0.67			
1.00			
1.33			
1.67			
2.00			
2.33			
2.67			
3.00			
3.33			
3.67			
4.00			
4.33			
4.67			
5.00			
5.33			
5.67			
6.00			
6.33			
6.67			
7.00			
7.33			
7.67			
8.00			
8.33			
8.67			
9.00			
9.33			
9.67			
10.00			
		∑ =	0.00

Notes:

This page intentionally left blank

to facilitate double-sided printing

A.5 Sand Filter Basin

Treatment Mechanisms	Filtration, Biofiltration
Maximum Tributary Drainage Area	25 acres
Other Names	Media Filter Basin, Pocket Filter Basin

A.5.1 General

A Sand Filter Basin (SFB) is a basin where the entire invert is constructed as a stormwater filter, using a sand bed above an underdrain system. Stormwater enters the SFB at its forebay where trash and sediment accumulate or through overland sheet flow. Overland sheet flow into the Sand Filter Basin is biofiltered through the vegetated side slopes or other pre-treatment. Flows pass into the sand filter surcharge zone and are gradually filtered through the underlying sand bed. The underdrain gradually dewaters the sand bed and discharges the filtered runoff to a nearby channel, swale, or storm drain.

Sand Filter (no forebay)
Photo courtesy of Colorado UDFCD

The primary advantage of the SFB is its effectiveness in removing pollutants where infiltration into the underlying soil is not practical, and where site conditions preclude the use of a bioretention facility. The primary disadvantage is a potential for clogging if silts and clays are allowed to flow into the SFB. In addition, this BMP's performance relies heavily on its being regularly and properly maintained.

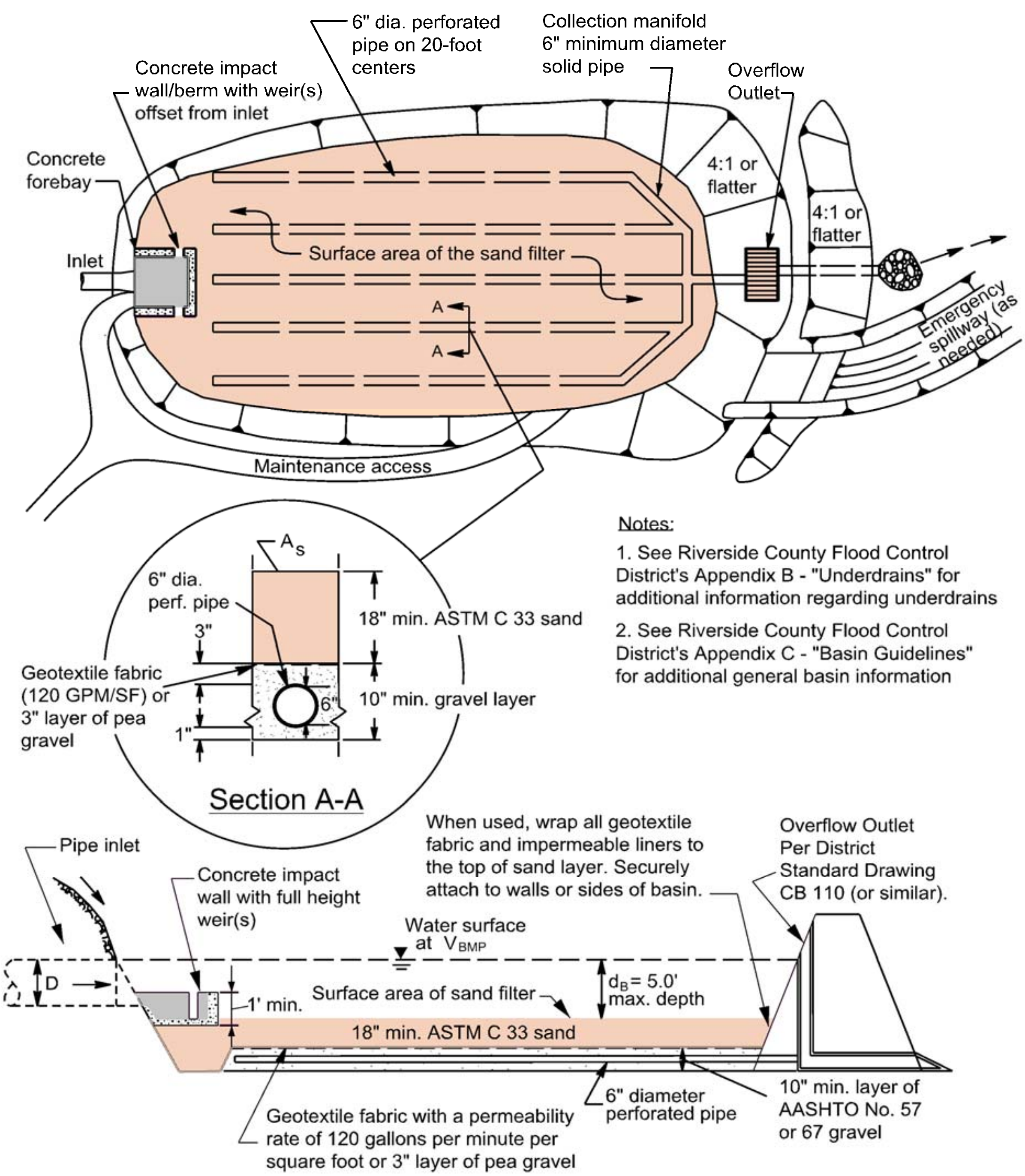
A.5.2 Siting Considerations

SFBs should be avoided where onsite configurations include a base flow and/or where this BMP would be put into operation while construction, grading or major landscaping activities are taking place in the tributary catchment. **This BMP has a flat surface area**, so it may be challenging to incorporate into steeply sloping terrain. SFBs should be set away from areas that could discharge fine sediments into the basin such as at the bottom of a slope. **Consult the local land use authority for additional guidance or requirements** (i.e., fencing, maintenance access, etc.)

Setbacks

The bottom of the sand filter should remain above the seasonal high groundwater level. Always consult your geotechnical engineer for additional site specific recommendations.

Figure 1 – Plan and Profile Views of SFB



Forebay

A concrete forebay shall be provided to reduce sediment clogging and to reduce erosion. The forebay shall have a design volume of at least 0.5% V_{BMP} and a minimum one foot high concrete splash wall. Full height notch-type weir(s), offset from the line of flow from the basin inlet to prevent short circuiting shall be used to outlet the forebay. It is recommended that two weirs be used and that they be located on opposite sides of the forebay (see Figure 1).

Underdrains

Underdrain piping shall consist of a manifold (collector) pipe with perforated lateral branching. The lateral branching conveys the filtered water to the manifold where it is discharged into the outlet structure. See Appendix C for additional information.

Overflow Structure

An overflow must be provided to drain volume in excess of V_{BMP} or to help drain the system if clogging were to occur. Overflows shall flow to an acceptable discharge point such as a downstream conveyance system. Overflows must be placed above the water quality capture volume and near the outlet of the system. The overflow structure shall be similar to the District's Standard Drawing CB 110; this drawing can be found at: http://rcflood.org/downloads/Standard%20Drawings/CB110.pdf.

A.5.3 Sand Filter Design and Sizing Criteria

Table 2 - Design and Sizing Criteria for SFBs

Design Parameter	Extended Detention Basin
Maximum Tributary Area	25 acres
Basin Design Volume	100% of V_{BMP}
Maximum Basin Depth	5 feet
Forebay Volume	0.5 % of V_{BMP}
Longitudinal Slope	0%
Transverse Slope (min.)	0%
Outlet Erosion Control	Energy dissipaters to reduce velocities

Note: The information contained in this BMP Factsheet is intended to be a summary of design considerations and requirements. Information herein may be superseded by other guidelines issued by the local land use authority.

Recommended Maintenance

Table 1 - Recommended Inspection and Maintenance Activities for SFBs

Schedule	Inspection and Maintenance Activity
Semi-monthly including just before the annual storm season and following rainfall events.	• Routine maintenance and inspection. • Remove debris and litter from the entire basin to minimize filter clogging and to improve aesthetics. • Check for obvious problems especially filter clogging and signs of long term ponding. Repair as needed. Address odor, insects, and overgrowth issues associated with stagnant or standing water in the basin bottom. There should be no long-term ponding water. • Check for erosion and sediment laden areas in the basin. Repair as needed. Clean forebay if needed. • Re-vegetate side slopes where needed.
Annually. If possible, schedule these inspections within 72 hours after a significant rainfall.	• Inspection of hydraulic and structural facilities. Examine the overflow outlet for clogging, the embankment and spillway integrity, and damage to any structural element. • Check side slopes and embankments for erosion, slumping and overgrowth. • Inspect the sand media at the filter drain to verify it is allowing acceptable infiltration. **Scarify the top three inches by raking the filter drain's sand surface annually.** • Check the filter drain underdrains for damage or clogging. Repair as needed. • Repair basin inlets, outlets, forebays, and energy dissipaters whenever damage is discovered. • No water should be present 48 hours after an event. No long-term standing water should be present at all. No algae formation should be visible. Correct problem as needed.
Every five years or sooner depending on the observed drain times (no more than 48 hours to empty the basin).	• Remove the top three inches of sand from the filter drain and backfill with three inches of new sand to return the sand layer to its original depth. When scarification or removal of the top three inches of sand is no longer effective, remove and replace sand filter layer.

A.5.4 Sand Filter Design Procedure

1. Enter the Tributary Area, A_{TRIB}
2. Enter the Design Capture Volume, V_{BMP}, determined from Worksheet 1 of this Handbook
3. SFB Geometry

   Determine the minimum sand filter area required. The filtration bed surface shall be flat with the maximum depth for the reservoir design volume no greater than five feet*. The reservoir design volume does not include the volume of the sand filter. No credit is given for voids in the sand layer toward the reservoir volume since the sand is part of the water quality filter and not a reservoir layer. The design storage volume shall equal 100% of V_{BMP}. The minimum sand filter area (As) of the basin's bottom shall be determined using the equation:

$$A_s = (V_{BMP}/ d_B)$$

Where:

V_{BMP} = design volume, ft^3
d_B = proposed basin depth, ft. (five feet maximum)

Once the basin side slopes, proposed basin depth, and depth of freeboard are entered, the spreadsheet will calculate the minimum total depth required to use this BMP. This is the depth from the top of the basin (including freeboard) down to the bottom of the underdrain gravel layer. This depth can be used to determine if enough vertical separation is available between the BMP and its outlet destination.

*Note: The five foot maximum depth equates to a minimum filter media infiltration rate of 1.25 inches per hour with a 48 hour drawdown time. Studies have shown that while initially most filter media will infiltrate at a much higher rate, it is not uncommon for that rate to decrease significantly over a very short period of time (Urbonas, 1996).

4. Enter the proposed surface area of the basin.

5. Forebay

   Provide a concrete forebay. Its volume shall be at least 0.5% V_{BMP} with a minimum one foot high concrete splash wall. Full-height notch-type weir(s) shall be used to outlet the forebay. The weir(s) must be offset from the line of flow from the basin inlet. It is recommended that two weirs be used and that they be located on opposite sides of the forebay (see Figure 1). Notches shall not be less than 1.5 inches in width.

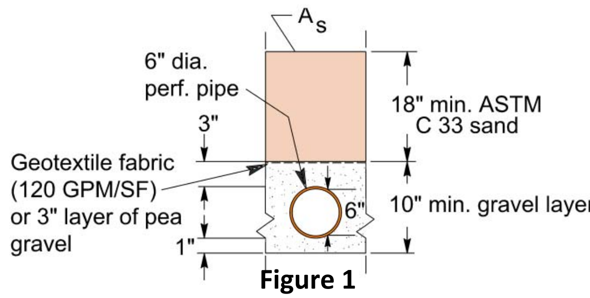


Figure 1

6. Filter Media

   Provide, as a minimum, an 18 inch layer of filter media (ASTM C-33 sand). Other filter media may be considered with sufficient supporting documentation. Where a medium level of removal efficiency is desired for nutrients, the depth of the sand layer must be increased to 36 inches.

7. Underdrains

   Underdrains shall be provided per the guidelines outlined in Appendix C.

Worksheet 7 – This worksheet is available on the District's Municipal Stormwater Management Program page for developers at: www.rcflood.org/NPDES/developers.aspx

Sand Filter Basin (SFB) - Design Procedure (Rev. 6-2014)	BMP ID	Legend:	Required Entries / Calculated Cells

Company Name: Date:

Designed by: County/City Case No.:

Design Volume

Total Tributary area	A_{TRIB} =	ac
Enter V_{BMP} determined from Section 4.3 of this Handbook	V_{BMP}=	ft^3

Basin Geometry

Basin side slopes (no steeper than 4:1)	z =	:1
Proposed basin depth (see Figure 1)	d_B =	ft
Depth of freeboard (if used)	d_{fb} =	ft
Minimum bottom surface area of basin ($As = V_{BMP}/d_B$)	A_s =	ft^2
Minimum total depth required (includes freeboard, filter media and subdrains)	d_{req} =	ft
Proposed Surface Area		ft^2

Forebay

Forebay volume (minimum 0.5% V_{BMP})	Volume =	ft^3
Forebay depth (height of berm/splashwall. 1 foot min.)	Depth =	ft
Forebay surface area (minimum)	Area =	ft^2
Full height notch-type weir	Width (W) =	in

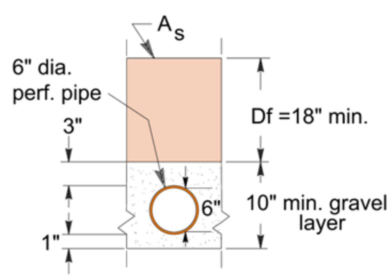
Filter Media

Description of filter media

Sand (ASTM C-33)

Other (Clarify in "Notes" below)

Media depth, df = inches



Underdrains

Diameter of perforated underdrain		in
Spacing of underdrains (maximum 20 feet on center)	OK	ft

Notes:

This page intentionally left blank

to facilitate double-sided printing

A.6 Permeable Pavement

Treatment Mechanisms	Infiltration, Evaporation
Maximum Drainage Area	10 acres
Other Names	Porous pavement, pervious pavement, pervious concrete, pervious asphalt, pervious gravel, pavement, cobblestone block, modular block, modular pavement

A.6.1 General

Permeable pavement can be either permeable asphalt and concrete surfaces, or permeable modular block. Unlike traditional pavements that are impermeable, permeable pavement reduce the volume and peak of stormwater runoff as well as mitigate pollutants from stormwater runoff, provided that the underlying soils can accept infiltration. Permeable pavement surfaces work best when they are designed to be flat or with gentle slopes. This factsheet discusses criteria that apply to infiltration designs.

The permeable surface is placed on top of a reservoir layer that holds the water quality stormwater volume, V_{BMP}. The water infiltrates from the reservoir layer into the native subsoil. Tests should be performed according to the Infiltration Testing Section in Appendix B to be able to use this design procedure.

In some circumstances, permeable pavement may be implemented on a project as a SRA. Where implemented as a SRA, the pavement is not considered a 'BMP' that would be required to be designed and sized per this Handbook. Where permeable pavement does not fit the specifications described by Section 2.1. The permeable pavement may be considered a BMP that must be sized according to this Hanbook Consult the local land use authority and the Whitewater River Region WQMP for any applicable requirements for designing and sizing permeable pavement installations.

A.6.2 Siting Considerations

Permeable pavement can be used in the same manner as concrete or asphalt in low traffic parking lots, playgrounds, walkways, bike trails, and sports courts. Most types of permeable pavement can be designed to meet Americans with Disabilities Act (ADA) requirements. Permeable pavement **should not** be used in the following conditions:

- 🛇 Downstream of erodible areas;
- 🛇 Downstream of areas with a high likelihood of pollutant spills;
- 🛇 Industrial or high vehicular traffic areas (25,000 or greater average daily traffic); and/or
- 🛇 Areas where geotechnical concerns, such as soils with low infiltration rates, would preclude the use of this BMP.

Sites with Impermeable Fire Lanes

Oftentimes, fire departments do not allow alternative pavement types including permeable pavement and require traditional impermeable surfaces for fire lanes. In this situation, it is acceptable to use an impermeable surface for the fire lane drive aisles and permeable pavement for the remainder of the parking lot.

Where impermeable fire lanes are used in the design, the impermeable surface must slope toward the permeable pavement, and the base layers shall remain continuous underneath the two pavement types, as shown in Figure 1. This continuous reservoir layer helps to maintain infiltration throughout the permeable pavement site and can still be considered as part of the total required storage area.

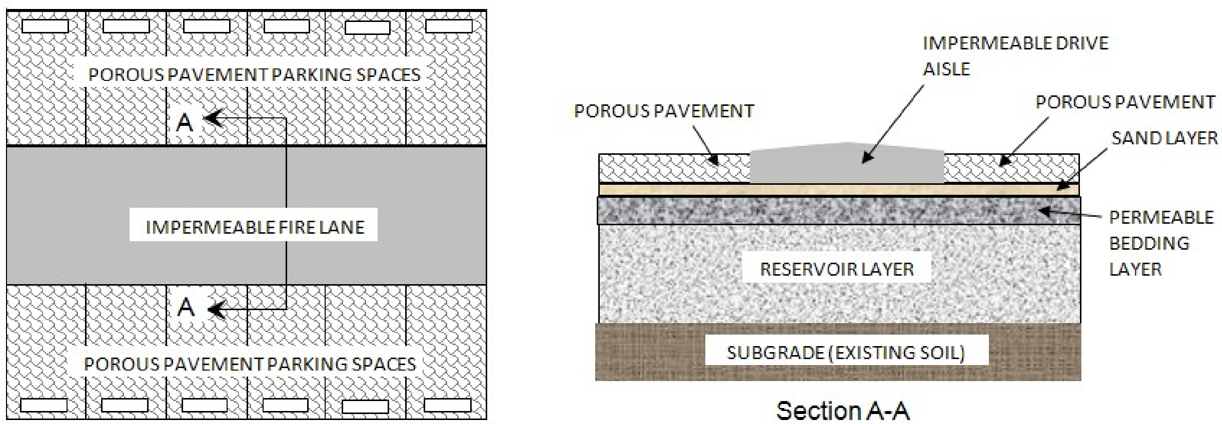


Figure 1: Impermeable Fire Lanes

Also, while a seal coat treatment may be used on the impermeable fire land, traditional seal coat treatments **shall not** be used on permeable pavement.

Setbacks

Always consult your geotechnical engineer for site-specific recommendations regarding setbacks for permeable pavement. Recommended setbacks are needed to protect buildings, walls, onsite wells, streams, and tanks.

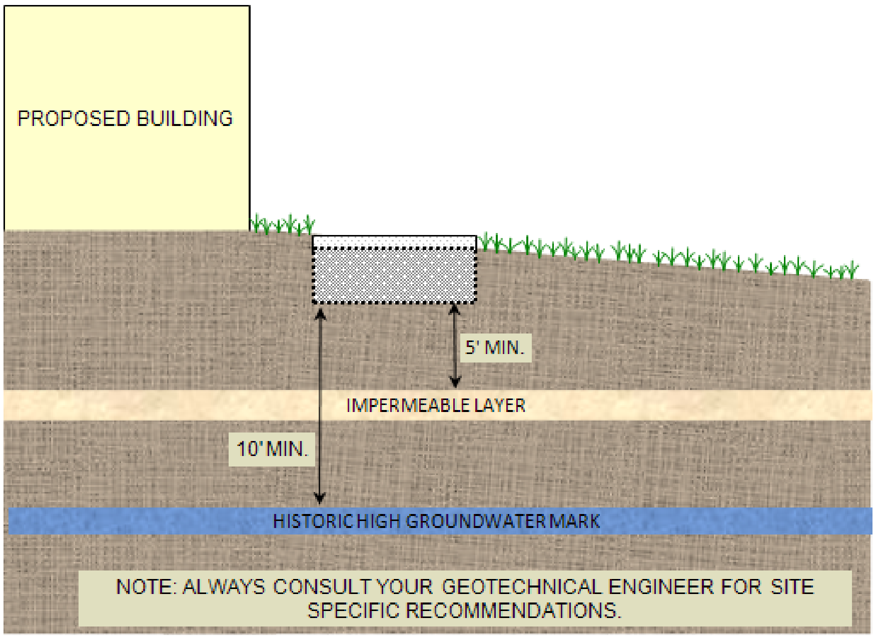


Figure 2: Permeable Pavement Setback Requirements

A minimum vertical separation of 10 feet is required from the bottom of the reservoir layer to the historic high groundwater mark (see Figure 2). A minimum vertical separation of five feet is required from the bottom of the reservoir layer to any impermeable layer in the soil. If the historic high groundwater mark is less than 10 feet below the reservoir layer section, or less than five feet from an impermeable layer, the infiltration design is not feasible.

A.6.3 Design and Sizing Criteria

To ensure that the pavement structural section is not compromised, a 48-hour drawdown time is utilized for this BMP.

Reservoir Layer Considerations

Even with proper maintenance, sediment will begin to clog the soil below the permeable pavement. Since the soil cannot be scarified or replaced, this will result in slower infiltration rates over the life of the permeable pavement. Therefore, the reservoir layer is limited to a maximum of 12 inches in depth to ensure that over the life of the BMP, the reservoir layer will drain in an adequate time.

***Note**: All permeable pavement BMP installations (not including permeable pavement as a self-retaining area) must be tested by the geotechnical engineer to ensure that the soils drain at a minimum allowable* *rate to ensure drainage. See the Infiltration Testing Section of this Handbook for specific details for the required testing and applied factors of safety.*

Sloping Pervious Paving

Ideally permeable pavement would be level; however, most sites will have a mild slope. If the tributary drainage area is too steep, the water may be flowing too fast when it approaches the permeable pavement, which may cause water to pass over the pavement instead of percolating and entering the reservoir layer. If the maximum slopes shown in Table 1 are complied with, it should address these concerns.

Table 1: Design Parameters for Permeable Paving

Design Parameter	Permeable Paving
Maximum slope of permeable pavement	3%
Maximum contributing area slope	5%

Regardless of the slope of the pavement surface design, the bottom of the reservoir layers shall be flat and level as shown in Figure 3. The design shown ensures that the water quality volume will be contained in the reservoir layer. A terraced design utilizing non- permeable check dams may be a useful option when the depth of gravel becomes too great as shown in Figure 4.

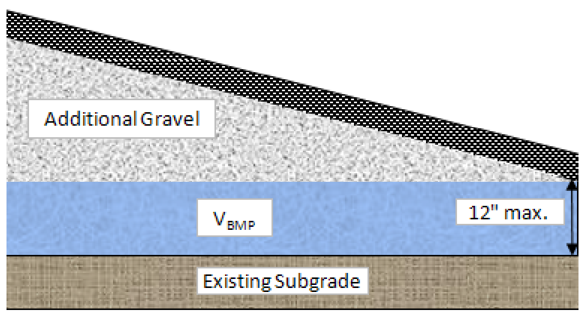


Figure 3: Sloped Cross-Sections for Permeable Pavement

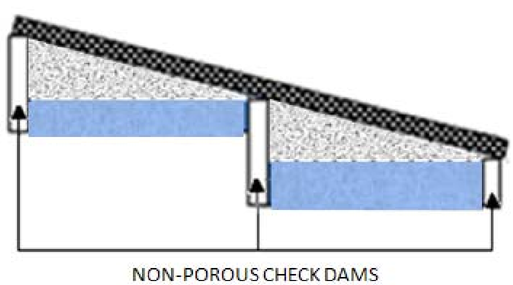


Figure 4: Permeable Pavement with Non-permeable Check Dams

In Figure 5, the bottom of the gravel reservoir layer is incorrectly sloped parallel to the pavement surface. Water would only be allowed to pond up to the lowest point of the BMP. Additional flows would simply discharge from the pavement. Since only a portion of the gravel layer can store water, this design would result in insufficient capacity.

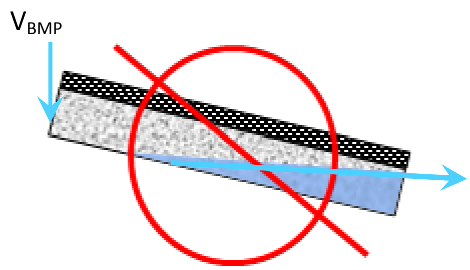


Figure 5: Incorrect Sloping of Permeable Pavement

To assure that the subgrade will empty within the 48 hour drawdown time, it is important that the maximum depth of 12 inches for the reservoir layer discussed in the design procedure is not exceeded. The value should be measured from the lowest elevation of the slope (Figure 4).

Minimum Surface Area

The minimum surface area required, A_S, is calculated by dividing the water quality volume, V_{BMP}, by the depth of water stored in the reservoir layer. The depth of water is found by multiplying the void ratio of the reservoir aggregate by the depth of the layer, b_{TH}. The void ratio of the reservoir aggregate is typically 40%; the maximum reservoir layer depth is 12 inches.

Sediment Control

A pretreatment BMP should be used for sediment control, and reduce the amount of sediment that enters the system and reduce clogging. The pretreatment BMP will also help to spread runoff flows, which allows the system to infiltrate more evenly. The pretreatment BMP must discharge to the surface of the pavement and not the subgrade. Grass swales may also be used as part of a treatment train with permeable pavement.

Liners and Filter Fabric

Always consult your geotechnical engineer for site specific recommendations regarding liners and filter fabrics. Filter fabric may be used around the edges of the permeable pavement; this will help keep fine sediments from entering the system. Unless recommended for the site, impermeable liners are not to be used below the sub-drain gravel layer.

Overflow

An overflow route is needed in the permeable pavement design to bypass storm flows larger than the V_{BMP} or in the event of clogging. Overflow systems must connect to an acceptable discharge point such as a downstream conveyance system.

Roof Runoff

Permeable pavement can be used to treat roof runoff, however, the runoff cannot be discharged beneath the surface of the pavement directly into the subgrade, as shown in Figure 6. Instead the pipe should empty on the surface of the permeable pavement as shown in Figure 7. A filter on the drainpipe should be used to help reduce the amount of sediment that enters the permeable pavement.

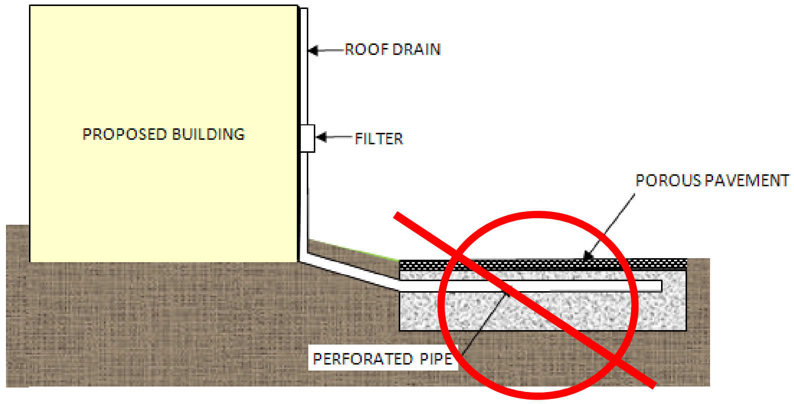


Figure 6: Incorrect Roof Drainage

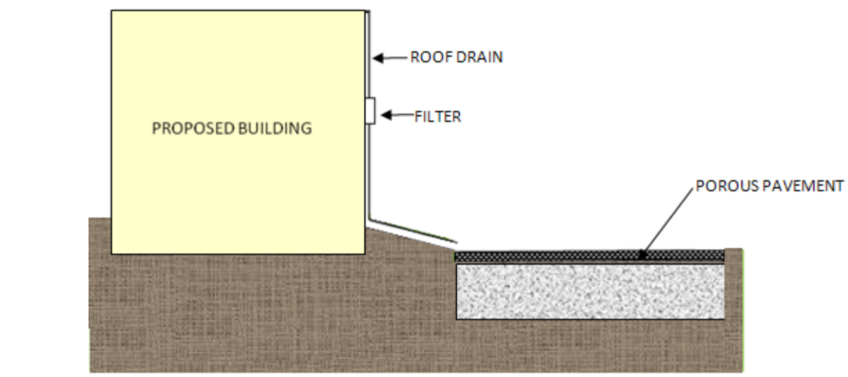


Figure 7: Correct Roof Runoff Drainage

Infiltration

Refer to the Infiltration Testing Section (Appendix B) in this Handbook for recommendations on testing for this BMP.

Pavement Section

The cross-section necessary for infiltration design of permeable pavement includes:

- The thickness of the layers of permeable pavement, sand and bedding layers depends on whether it is permeable modular block or permeable pavement. A licensed geotechnical or civil engineer is required to determine the thickness of these upper layers appropriate for the pavement type and expected traffic loads.

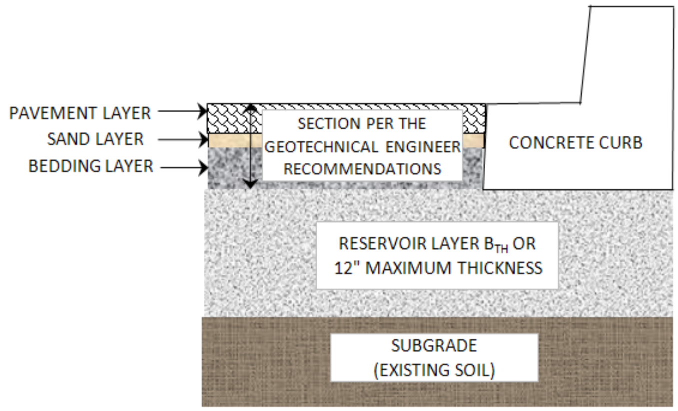


Figure 8: Infiltration Cross-Section

- A 12” maximum reservoir layer consisting of AASHTO #57 gravel vibrated in place or equivalent with a minimum of 40% void ratio.

Table 1 - Inspection and Maintenance Schedule –Modular Block

Schedule	Activity
Ongoing	• Keep adjacent landscape areas maintained. Remove clippings from landscape maintenance activities. • Remove trash and debris.
Utility trenching and other pavement repairs	• Remove and reset modular blocks, structural section and reservoir layer as needed. Replace damaged blocks in-kind. • Do not pave repaired areas with impermeable surfaces.
After storm events	• Inspect areas for ponding.
2-3 times per year	• Sweep to reduce the chance of clogging.
As needed	• Sand between pavers may need to be replaced if infiltration capacity is lost.

Table 2 - Inspection and Maintenance Schedule –Permeable Concrete/Asphalt

Schedule	Activity
Ongoing	• Keep adjacent landscape areas maintained. Remove clippings from landscape maintenance activities. • Remove trash and debris.
Utility trenching and other pavement repairs	• Replace structural section and reservoir layer in-kind. • Re-pave using permeable concrete/asphalt. Do not pave repaired areas with impermeable surfaces.
After storm events	• Inspect areas for ponding
2-3 times per year	• Vacuum the permeable pavement to reduce the chance of clogging.
As needed	• Remove and replace damaged or destroyed permeable pavement.

A.6.4 Design Procedure Permeable Paving

1. Enter the Tributary Drainage Area, A_{TRIB}.
2. Enter the Design Volume, V_{BMP}, determined from Worksheet 1 of this Handbook.
3. Enter the reservoir layer depth, b_{TH} for the proposed permeable pavement. The reservoir layer maximum depth is 12 inches.
4. Calculate the Minimum Surface Area, A_S, required.

$$A_S(\text{ft}^2) = \frac{\text{VBMP (ft}^3)}{(0.4 \text{ x } b_{TH} \text{ (in)}) \ / \ 12 \ (^{\text{in}}/_{\text{ft}})}$$

   Where, the porosity of the gravel in the reservoir layer is assumed to be 40%.

5. Enter the proposed surface area and ensure that this is equal to or greater than the minimum surface area required.
6. Enter the dimensions, per the geotechnical engineer's recommendations, for the pavement cross- section. The cross-section includes a pavement layer, usually a sand layer and a permeable bedding layer. Then add this to the maximum thickness of the reservoir layer to find the total thickness of the BMP.
7. Enter the slope of the top of the permeable pavement. The maximum slope is 3%.
8. Enter whether sediment control was provided.
9. Enter whether the geotechnical approach is attached.
10. Describe the surfaces surrounding the pervious pavement. It is preferred that a vegetation buffer is used around the pervious pavement.
11. Check to ensure that vertical setbacks are met. There should be a minimum of 10 feet between the bottom of the BMP and the top of the high groundwater table, and a minimum of five feet between the reservoir layer the top of the impermeable layer.

Worksheet 8 – This worksheet is available on the District's Municipal Stormwater Management Program page for developers at: www.rcflood.org/NPDES/developers.aspx

Permeable Pavement - Design Procedure (Rev. 06-2014)	BMP ID	Legend:	Required Entries / Calculated Cells
Company Name:		Date:	
Designed by:		County/City Case No.:	

Design Volume

Enter the area tributary to this feature — A_{TRIB}= ______ acres

Enter V_{BMP} determines from Section 4.3 of this Handbook — V_{BMP}= ______ ft^3

Permeable Pavement Surface Area

Reservoir Layer Depth, b_{TH} — b_{TH}= ______ inches

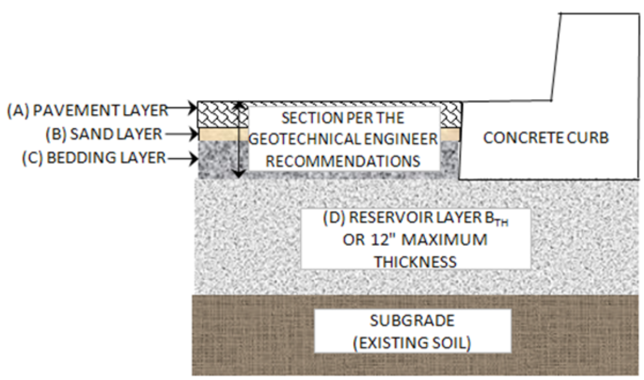
Minimum Surface Area Required, A_S

$$A_S(ft) = \frac{V_{BMP}(ft^3)}{(0.4 \times b_{TH}(in)) / 12(in/ft)}$$

A_S= ______ ft^2

Proposed Surface Area = ______ ft^2

Permeable Pavement Cross Section



Per the Geotechnical	(A)		in
Engineer's	(B)		in
Recommendations	(C)		in
Reservoir Layer	(D)		in
Total Permeable Pavement Section			in
Slope of Permeable Pavement			%

Sediment Control Provided? (Use pulldown) ______

Geotechnical report attached? (Use pulldown) ______

Describe Surrounding Landscaping: ______

Notes: ______

If the Permeable Pavement has been designed correctly, there should be no error messages on the spreadsheet.

This page intentionally left blank

to facilitate double-sided printing

A.7 Infiltration Basin

Treatment Mechanisms	Infiltration, Evapotranspiration (when vegetated), Evaporation, Sedimentation
Maximum Drainage Area	50 acres
Other Names	Biofiltration Basin, Retention Basin

A.7.1 General

An infiltration basin is a flat earthen basin designed to capture the design capture volume, V_{BMP}. The stormwater infiltrates through the bottom of the basin into the underlying soil over a 48 hour drawdown period. Flows exceeding V_{BMP} must discharge to a downstream conveyance system. Trash and sediment accumulate within the forebay as stormwater passes into the basin. Infiltration basins are highly effective in removing all targeted pollutants from stormwater runoff.

Figure 1 – Infiltration Basin

A.7.2 Siting Consideration

Important Note: It is recommended that the design procedure for infiltration basins described in this Handbook not be used when a project is subject to a local land use authority onsite retention ordinance. Table 4 in Section 6.1 lists the Whitewater River Region local land use authorities with onsite retention requirements. The use of infiltration basins may be restricted by concerns over ground water contamination, soil permeability, and clogging at the site. See the applicable WQMP for any specific feasibility considerations for using infiltration BMPs. Where this BMP is being used, the soil beneath the basin must be thoroughly evaluated in a geotechnical report since the underlying soils are critical to the basin's long-term performance. To protect the basin from erosion, the sides and bottom of the basin must be vegetated, preferably with native or low water use plant species.

In addition, these basins may not be appropriate for the following site conditions:

- Industrial sites or locations where spills of toxic materials may occur;
- Sites with very low soil infiltration rates;
- Sites with high groundwater tables or excessively high soil infiltration rates, where pollutants can affect ground water quality;
- Sites with un-stabilized soil or construction activity upstream;
- On steeply sloping terrain; and/or
- Infiltration basins located in a fill condition should refer to Appendix B of this Handbook for details on special requirements/restrictions.

A.7.3 Infiltration Basin Design Criteria

Table 1 - Infiltration Basin Design and Sizing Criteria

Design Parameter	Infiltration Basin
Design Volume	V_{BMP}
Forebay Volume	0.5% V_{BMP}
Drawdown Time (maximum)	48 hours
Maximum Tributary Area	50 acres
Minimum Infiltration Rate	Must be sufficient to drain the basin within the required Drawdown time over the life of the BMP.
Maximum Depth	5 feet
Spillway Erosion Control	Energy dissipaters to reduce velocities
Basin Slope	0%
Freeboard (minimum)	1 foot
Historic High Groundwater Setback (min)	10 feet
Bedrock/Impermeable Layer Setback (min)	5 feet
Tree Setbacks	Mature tree drip line must not overhang the basin
Setback from wells, tanks or springs	100 feet
Setback from foundations	As recommended in Geotechnical Report
Embankment Side Slope (H:V)	4:1 or flatter inside slope/ 3:1 or flatter outside slope (without retaining walls), or as approved by the local land use authority
Maintenance Access Ramp Slope (H:V)	10:1 or flatter, or as approved by the local land use authority
Vegetation	Side slopes and bottom (may require irrigation in the summer)

Note: The information contained in this BMP Factsheet is intended to be a summary of design considerations and requirements. Information herein may be superseded by other guidelines issued by the local land use authority.

Setbacks

Always consult your geotechnical engineer for site-specific recommendations regarding setbacks for infiltration trenches. Recommended setbacks are needed to protect buildings, existing trees, walls, onsite or nearby wells, streams, and tanks. Setbacks should be considered early in the design process since they can affect where infiltration facilities may be placed and how deep they are allowed to be. For instance, depth setbacks can dictate fairly shallow facilities that will have a larger footprint, and in some cases, may make an infiltration basin infeasible. In that instance, another BMP must be selected.

Infiltration basins typically must be set back:

- 10 feet from the historic high groundwater (measured vertically from the bottom of the basin, as shown in Figure 2)
- 5 feet from bedrock or impermeable surface layer (measured vertically from the bottom of the basin, as shown in Figure 2)
- From all existing mature tree drip lines as indicated in Figure 2 (to protect their root structure)
- 100 feet horizontally from wells, tanks or springs

Setbacks to walls and foundations must be included as part of the Geotechnical Report.

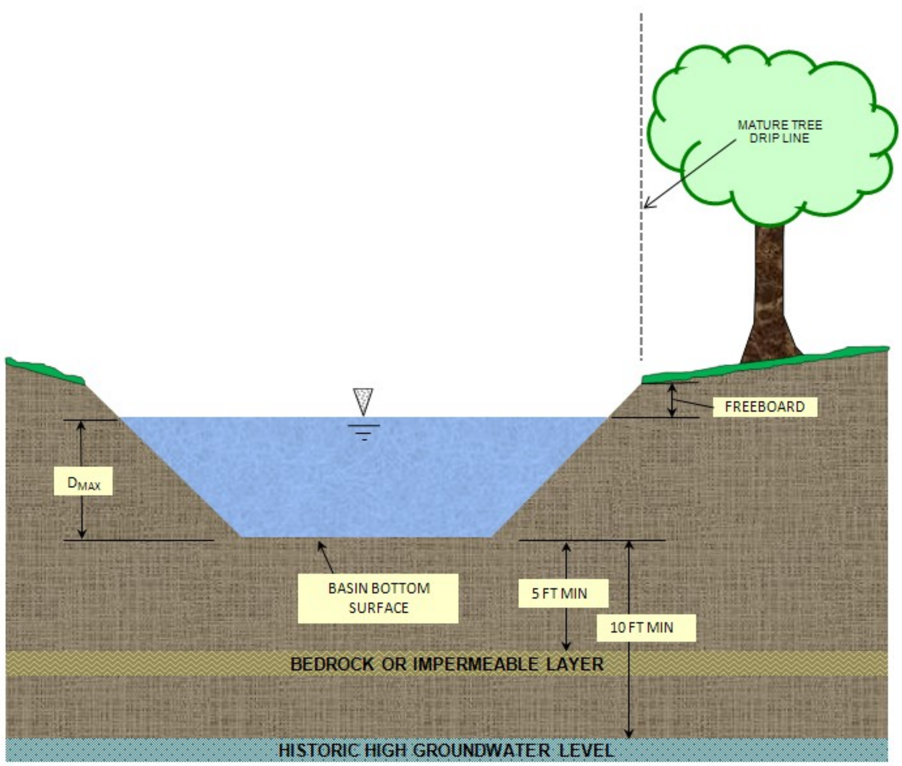


Figure 2 – Setback Requirements

Forebay

A concrete forebay should be provided to reduce sediment clogging and to reduce erosion. The forebay should have a design volume of at least 0.5% V_{BMP} and a minimum one foot high concrete splash wall/berm. Full height notch-type weir(s), offset from the line of flow from the basin inlet to prevent short circuiting, shall be used to outlet the forebay. It is recommended that two weirs be used and that they be located on opposite sides of the forebay (see Figure 2).

Overflow

Flows exceeding V_{BMP} must discharge to an acceptable downstream conveyance system. Where an adequate outlet is present, an overflow structure may be used. Where an embankment is present, an emergency spillway may be used instead. Overflows must be placed just above the design water surface for V_{BMP} and be near the outlet of the system. The overflow structure shall be similar to the District's Standard Drawing CB 110, which can be found at: http://rcflood.org/downloads/Standard%20Drawings/CB110.pdf.

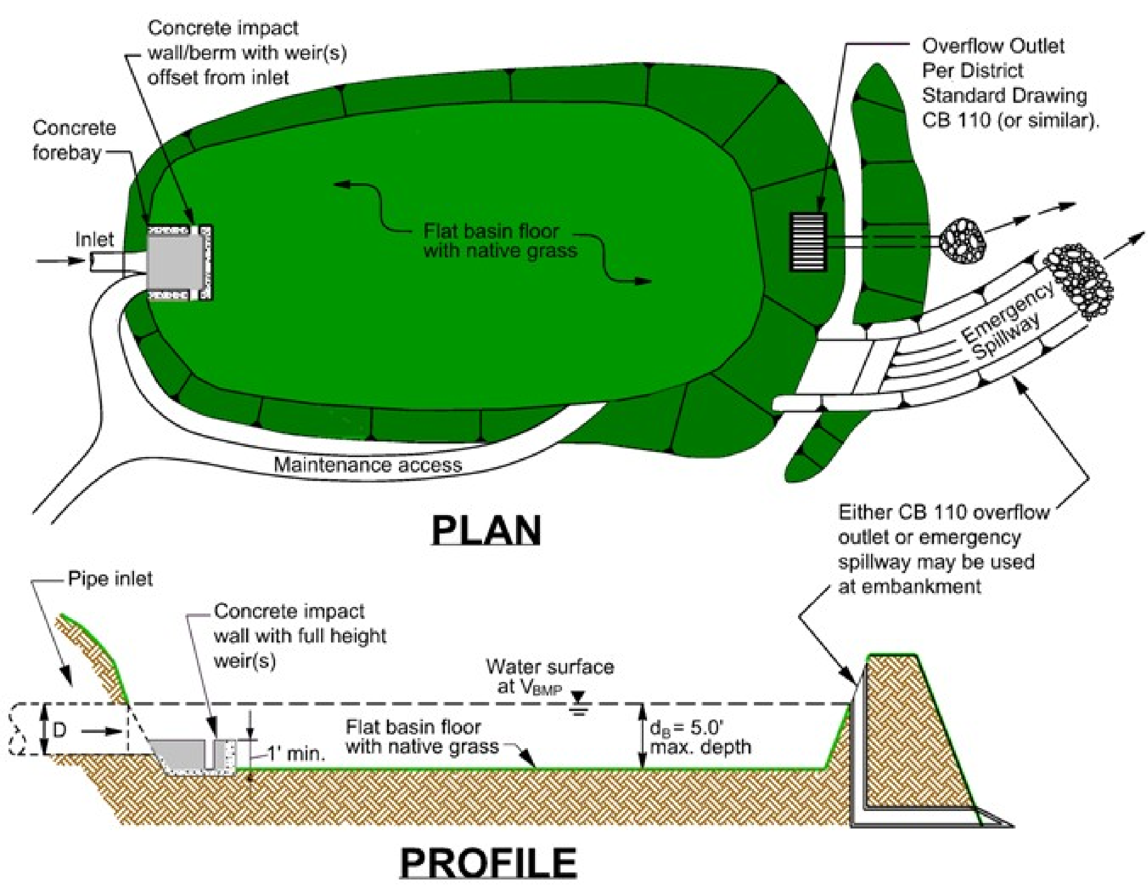


Figure 3 – Infiltration Basin

Landscaping Requirements

Basin vegetation provides erosion protection, improves sediment removal, and assists in allowing infiltration to occur. The basin surface and side slopes should be planted with desert appropriate native landscaping. Proper landscape management is also required to ensure that the vegetation does not contribute to water pollution through pesticides, herbicides, or fertilizers. Landscaping should be in accordance with County of Riverside Ordinance 859, or other guidelines issued by the local land use authority. See table in Section 6.2 for local land use authority requirements.

Maintenance

Normal maintenance of an infiltration basin includes the maintenance of landscaping, debris and trash removal from the surface of the basin, and tending to problems associated with standing water (vectors, odors, etc.). Significant ponding, especially more than 48 hours after an event, may indicate that the basin surface is no longer providing sufficient infiltration and requires aeration.

Table 2 - Inspection and Maintenance Schedule

Schedule	Inspection and Maintenance Activity
Ongoing including just before annual storm seasons and following rainfall events.	• Maintain vegetation as needed. Use of fertilizers, pesticides and herbicides should be strenuously avoided to ensure they don't contribute to water pollution. If appropriate native plant selections and other IPM methods are used, such products shouldn't be needed. If such projects are used, o Products shall be applied in accordance with their labeling, especially in relation to application to water, and in areas subjected to flooding. o Fertilizers should not be applied within 15 days before, after, or during the rainy season. • Remove debris and litter from the entire basin to minimize clogging and improve aesthetics. • Check for obvious problems and repair as needed. Address odor, insects, and overgrowth issues associated with stagnant or standing water in the basin bottom. There should be no long-term ponding water. • Check for erosion and sediment laden areas in the basin. Repair as needed. Clean forebay if needed. • Re-vegetate side slopes where needed.
Annually. If possible, schedule these inspections within 48 hours after a significant rainfall.	• Inspection of hydraulic and structural facilities. Examine the inlet for blockage, the embankment and spillway integrity, as well as damage to any structural element. • Check for erosion, slumping and overgrowth. Repair as needed. • Check basin depth for sediment build up and reduced total capacity. Scrape bottom as needed and remove sediment. Restore to original cross-section and infiltration rate. Replant basin vegetation. • Verify the basin bottom is allowing acceptable infiltration. Use a disc or other method to aerate basin bottom only if there is actual significant loss of infiltrative capacity, rather than on a routine basis[1]. • No water should be present 48 hours after an event. No long term standing water should be present at all. No algae formation should be visible. Correct problem as needed.

A.7.4 Infiltration Basin Design Procedure

1. Find the Design Volume, V_{BMP}.
   a) Enter the Tributary Drainage Area, A_{TRIB}.
   b) Enter the Design Volume, V_{BMP}, determined from **Worksheet 1** of this Handbook.
2. Determine the Maximum Depth.
   a) Enter the infiltration rate. The infiltration rate should be established as described in Appendix B: "Infiltration Testing Guidelines".
   b) Enter the design Factor of Safety from Table 1 in Appendix B: "Infiltration Testing Guidelines".
   c) The spreadsheet will determine D_1, the maximum allowable depth of the basin based on the infiltration rate along with the maximum drawdown time (48 hours) and the Factor of Safety.

   $$D_1 = [(t) \times (I)] / 12(FS)$$

   Where I = site infiltration rate (in/hr)

   FS = safety factor

   t = drawdown time (maximum 48 hours)

   d) Enter the depth of freeboard.
   e) Enter the depth to the historic high groundwater level measured from the top of the basin.
   f) Enter the depth to the top of bedrock or other impermeable layer measured from the finished grade.
   g) The spreadsheet will determine D_2, the total basin depth (including freeboard, if used) of the basin, based on restrictions to the depth by groundwater and an impermeable layer.

   D_2 = Depth to groundwater – (10 + freeboard) (ft);

   or

   D_2 = Depth to impermeable layer – (five + freeboard) (ft)

   Whichever is least.

   h) The spreadsheet will determine the maximum allowable effective depth of basin, D_{MAX}, based on the smallest value between D_1 and D_2. D_{MAX} is the maximum depth of water only and does not include freeboard. D_{MAX} shall not exceed five feet.
3. Basin Geometry
   a) Enter the basin side slopes, z (no steeper than 4:1, unless allowed by local land use authority).
   b) Enter the proposed basin depth, d_B excluding freeboard.
   c) The spreadsheet will determine the minimum required surface area of the basin: $A_s = V_{BMP}/d_B$

Where A_s = minimum area required (ft^2)

V_{BMP} = volume of the infiltration basin (ft^3)

d_B= proposed depth not to exceed maximum allowable depth, D_{MAX} (ft)

d) Enter the proposed bottom surface area. This area should not be less than the minimum required surface area.

4. Forebay

A concrete forebay with a design volume of at least 0.5% V_{BMP} and a minimum one foot high concrete splash wall shall be provided. Full-height rectangular weir(s) should be used to outlet the forebay. The weir(s) must be offset from the line of flow from the basin inlet. It is recommended that two weirs be used and that they be located on opposite sides of the forebay (see Figure 2).

a) The spreadsheet will determine the minimum required forebay volume based on 0.5% V_{BMP}.

b) Enter the proposed depth of the forebay berm/splash wall (one foot minimum).

c) The spreadsheet will determine the minimum required forebay surface area.

d) Enter the width of rectangular weir to be used (minimum 1.5 inches). Weir width should be established based on a five minute drawdown time.

Worksheet 9 – This worksheet is available at the District's Municipal Stormwater Management Program page for developers at: www.rcflood.org/NPDES/developers.aspx

Infiltration Basin - Design Procedure (Rev. 06-2014)	BMP ID	Legend:	Required Entries / Calculated Cells
Company Name:		Date:	
Designed by:		County/City Case No.:	
Design Volume			
a) Tributary Drainage Area (BMP subarea)		A_{TRIB} =	acres
b) Enter V_{BMP} determined from Section 4.3 of this Handbook		V_{BMP}=	ft^3
Maximum Depth			
a) Infiltration rate		I =	in/hr
b) Factor of Safety (See Table 1, Appendix B: "Infiltration Testing" from this BMP Handbook)		FS =	
c) Calculate D_1	$D_1 = \frac{I\ (in/hr) \times 48\ hrs}{12\ (in/ft) \times FS}$	D_1 =	ft
d) Enter the depth of freeboard (at least 1 ft)			ft
e) Enter depth to historic high ground water (measured from **top** of basin)			ft
f) Enter depth to top of bedrock or impermeable layer (measured from **top** of basin)			ft
g) D_2 is the smaller of: Depth to groundwater - (10 ft + freeboard) and Depth to impermeable layer - (5 ft + freeboard)		D_2 =	ft
h) D_{MAX} is the smaller value of D_1 and D_2 but shall not exceed 5 feet		D_{MAX} =	ft
Basin Geometry			
a) Basin side slopes (no steeper than 4:1)		z =	:1
b) Proposed basin depth (excluding freeboard)		d_B =	ft
c) Minimum bottom surface area of basin ($A_S = V_{BMP}/d_B$)		A_S =	ft^2
d) Proposed Design Surface Area		A_D =	ft^2
Forebay			
a) Forebay volume (minimum 0.5% V_{BMP})		Volume =	ft^3
b) Forebay depth (height of berm/splashwall. 1 foot min.)		Depth =	ft
c) Forebay surface area (minimum)		Area =	ft^2
d) Full height notch-type weir		Width (W) =	in
Notes:			

A.8 Infiltration Trench

Treatment Mechanisms	Infiltration, Evapotranspiration (when vegetated), Evaporation
Maximum Drainage Area	10-acres
Other Names	None

A.8.1 General

Infiltration trenches are shallow excavated areas that are filled with rock material to create a subsurface reservoir layer. The trench is sized to store the design capture volume, V_{BMP}, in the void space between the rocks. Over a period of 48 hours, the stormwater infiltrates through the bottom of the trench into the surrounding soil. Infiltration basins are highly effective in removing all targeted pollutants from stormwater runoff.

A.8.2 Siting Considerations

The use of infiltration trenches may be restricted by concerns over groundwater contamination, soil permeability, and clogging at the site. Where this BMP is being used, the soil beneath the basin must be thoroughly evaluated in a geotechnical report since the underlying soils are critical to the basin's long-term performance. These basins may not be appropriate for the following site conditions:

- Industrial sites or locations where spills of toxic materials may occur.
- Sites with very low soil infiltration rates.
- Sites with high groundwater tables or excessively high soil infiltration rates, where pollutants can affect groundwater quality.
- Sites with un-stabilized soil or construction activity upstream.
- On steeply sloping terrain.
- Infiltration trenches located in a fill condition should refer to Appendix B of this Handbook for details on special requirements/restrictions.

This BMP has a flat surface area, so it may be challenging to incorporate into steeply sloping terrain.

A.8.3 Infiltration Trench Design Criteria

Table 1 - Infiltration Trench Design and Sizing Criteria

Design Parameter	Design Criteria
Design Volume	V_{BMP}
Design Drawdown time	48 hours
Maximum Tributary Drainage Area	10 acres
Maximum Trench Depth	8 feet
Width to Depth Ratio	Width must be greater than depth
Reservoir Rock Material	AASHTO #3 or 57 material or a clean, washed aggregate one to 3 inch diameter equivalent
Filter Strip Width	Minimum of five feet in the direction of flow for all areas draining to trench
Filter Strip Slope	Max slope = 1%
Filter Strip Materials	Mulch or desert appropriate native landscaping (non-mowed variety preferred)
Historic High Groundwater Mark	10 feet or more below bottom of trench
Bedrock/Impermeable Layer Setback	Five feet or more below bottom of trench
Tree Setbacks	Mature tree drip line must not overhang the trench
Trench Lining Material	As recommended in Geotechnical Report

Setbacks

Always consult your geotechnical engineer for site-specific recommendations regarding setbacks for infiltration trenches. Recommended setbacks are needed to protect buildings, walls, onsite or nearby wells, streams, and tanks. Setbacks should be considered early in the design process as they affect where infiltration facilities may be placed and how deep they are allowed to be. For instance, depth setbacks can dictate fairly shallow facilities that will have a larger footprint and, in some cases, may make an infiltration trench infeasible. In that instance, another BMP must be selected.

In addition to setbacks recommended by the geotechnical engineer, infiltration trenches should be setback:

- 10 feet from the historic high groundwater mark (measured vertically from the bottom of the trench, as shown in Figure 1)
- Five feet from bedrock or impermeable surface layer (measured vertically from the bottom of the trench, as shown in Figure 1)
- From all mature tree drip lines as indicated in Figure 1
- 100 feet horizontally from wells, tanks or springs

Setbacks to walls and foundations should be included as part of the Geotechnical Report.

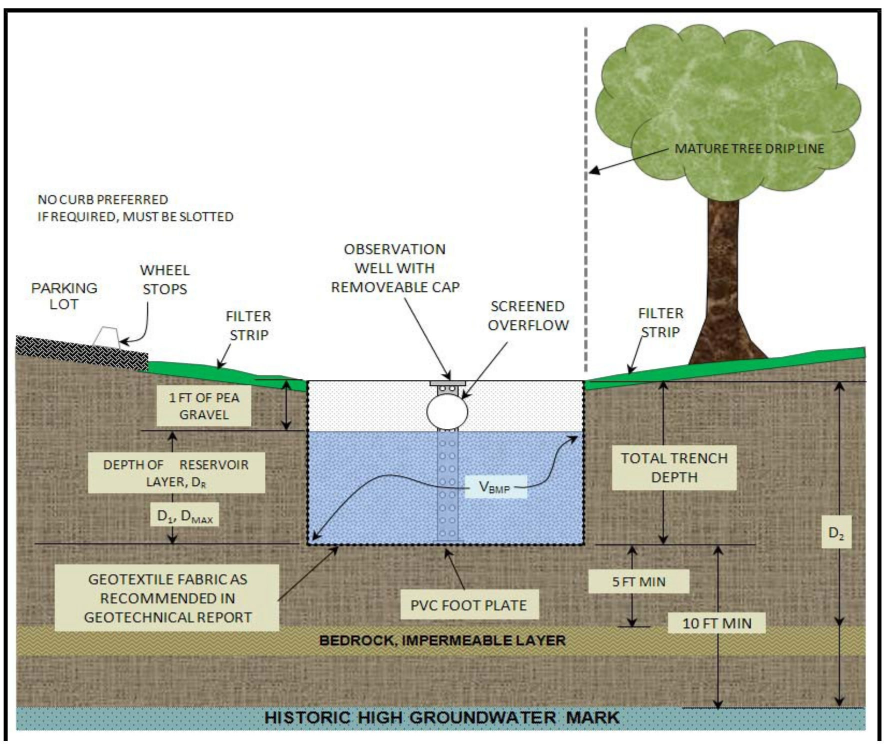


Figure 1 Section View of an Infiltration Trench

Sediment Control

Infiltration BMPs have the risk of becoming plugged over time. To prevent this, sediment must be removed before stormwater enters the trench. Both sheet and concentrated flow types have requirements that should be considered in the design of an infiltration trench.

When sheet type flows approach the trench along its length (as illustrated in Figure 2), a vegetated filter strip should be placed between the trench and the upstream drainage area. The filter strip must be a minimum of five feet wide and planted with desert appropriate native landscaping or covered with mulch.

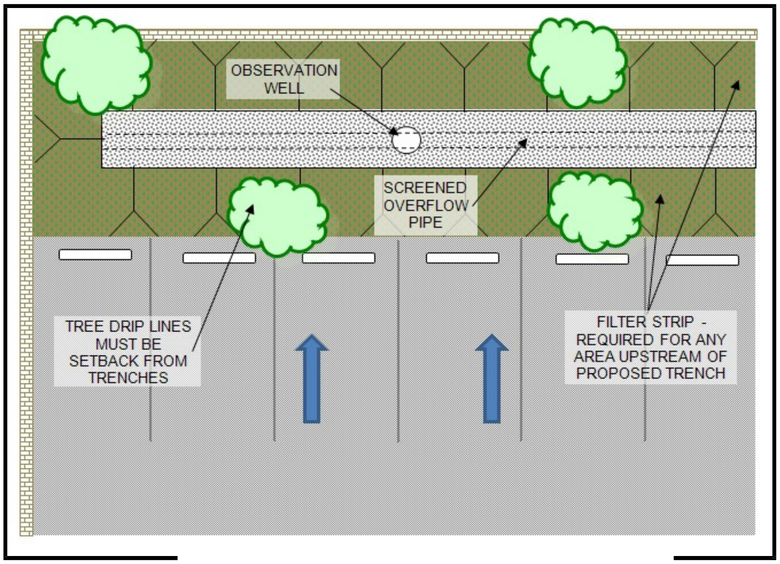


Figure 2 Plan View, Sheet Type Flows

Concentrated flows require a different approach. A 2004 Caltrans BMP Retrofit Report found that flow spreaders recommended in many water quality manuals are ineffective in distributing concentrated flows. As such, concentrated flows should either be directed toward a traditional vegetated swale (as shown on the right side of Figure 3) or to catch basin filters that can remove litter and sediment. Catch basins must discharge runoff as surface flow above the trench; they cannot outlet directly into the reservoir layer of the infiltration trench. If catch basins are used, the short and long term costs of the catch basin filters should be considered.

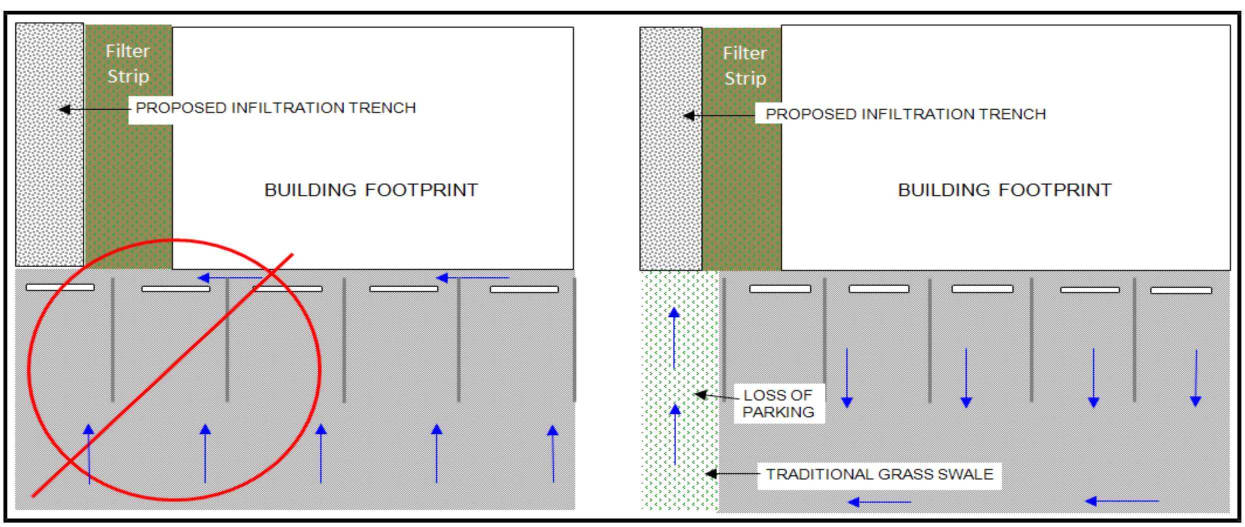


Figure 3 Plan View, Concentrated Flows

Geotechnical Report

A geotechnical report should be included for all infiltration trenches. Appendix B of this Handbook entitled “Infiltration Testing Guidelines”, details which types of infiltration tests are acceptable and how many tests or boring logs must be performed. A geotechnical report must be submitted in support of all infiltration trenches. Setbacks to walls and foundations must be included in the Geotechnical Report.

Observation Wells

One or more observation wells should be provided. The observation well consists of a vertical section of perforated pipe, four to six inches in diameter, installed flush with top of trench on a foot plate and have a locking, removable cap.

Overflow

An overflow route is needed to bypass storm flows larger than the V_{BMP} or in the event of clogging. Overflow systems must connect to an acceptable discharge point such as a downstream conveyance system.

Maintenance Access

Normal maintenance of an infiltration trench includes maintenance of the filter strip as well as debris and trash removal from the surface of the trench and filter strip. More substantial maintenance requiring vehicle access may be required every five to 10 years. Vehicular access along the length of the swale should be provided to all infiltration trenches. It is preferred that trenches be placed longitudinally along a street or adjacent to a parking lot area. These conditions have high visibility which makes it more likely that the trench will be maintained on a regular basis.

Table 2 - Inspection and Maintenance Schedule

Schedule	Inspection and Maintenance Activity
Every two weeks, or as often as necessary to maintain a pleasant appearance	- Maintain adjacent landscaped areas. Remove clippings from landscape maintenance activities. - Remove trash & debris
3 days after Major Storm Events	- Check for surface ponding. If ponding is only above the trench, remove, wash and replace pea gravel. May be needed every five to 10 years. - Check observation well for ponding. If the trench becomes plugged, remove rock materials. Provide a fresh infiltration surface by excavating an additional two to four inches of soil. Replace the rock materials.

A.8.4 Infiltration Trench Design Procedure

1. Enter the area tributary to the trench, maximum drainage area is 10 acres.
2. Enter the Design Volume, V_{BMP}, determined from **Worksheet 1** of this Handbook.
3. Enter the site infiltration rate, found in the geotechnical report.
4. Enter the factor of safety from Table 1 of Appendix B, Infiltration Testing Guidelines.
5. Determine the maximum reservoir layer depth, D_{MAX}. The value is obtained by taking the smaller of two depth equations but may never exceed 8 feet. The first depth, D_1 is related to the infiltration rate of the soil. The second depth, D_2, is related to required setbacks to groundwater, bedrock/impermeable layer. These parameters are shown in Figure 1.

   Calculate D_1.

   $$D_1 = \frac{I\left(^{in}/_{hr}\right) x\ 48\ (hrs)}{12\ \left(^{in}/_{hr}\right)\ x\ ^{n}/_{100}\ x\ FS}$$

   Where:

   I = site infiltration rate (in/hr), found in the geotechnical report

   FS = factor of safety, refer to Appendix B - Infiltration Testing

   n = porosity of the trench material, 40%

   Calculate D_2.
   Enter the depth to the seasonal high groundwater and bedrock/impermeable layer measured from the finished grade. The spreadsheet checks the minimum setbacks shown in Figure 1 and selects the smallest value. The equations are listed below for those doing hand calculations.

   Minimum Setbacks (includes 1 foot for pea gravel):
   = Depth to historic high groundwater mark - 11 feet
   = Depth to impermeable layer - 6 feet

   D_2 is the smaller of the two values.

   D_{MAX} is the smaller value of D_1 and D_2, and must be less than or equal to eight feet.
6. Enter the proposed reservoir layer depth, D_R. The value must be no greater than D_{MAX}.

7. Find the required surface area of the trench, A_S. Once D_R is entered, the spreadsheet will calculate the corresponding depth of water and the minimum surface area of the trench.

$$\text{Design } d_W = D_R x \left({}^{n}/_{100}\right) \qquad A_s = \frac{V_{BMP}}{\text{Design } d_w}$$

Where:

A_S = minimum area required (ft^2)

V_{BMP} = BMP storage volume (ft^3)

Design d_W = depth of water in reservoir layer (ft)

8. Enter the proposed design surface area; it must be greater than the minimum surface area.
9. Calculate the minimum trench width. This is to ensure that EPA's Class V Injection well status is not triggered. The total trench depth (shown in Figure 1) includes the upper foot where the overflow pipe is located. The minimum surface dimension is DR + one foot.

Additional Items

The following items detailed in the preceding sections should also be addressed in the design.

- Sediment Control
- Geotechnical Report
- Observation well(s)
- Overflow

Worksheet 10 – This worksheet is available on the District's Municipal Stormwater Management Program page for developers at: www.rcflood.org/NPDES/developers.aspx

Infiltration Trench - Design Procedure (Rev. 06-2014)	BMP ID	Legend:	Required Entries / Calculated Cells
Company Name:		Date:	
Designed by:		County/City Case No.:	

Design Volume

Enter the area tributary to this feature, Max = 10 acres	A_{TRIB}=		acres
Enter V_{BMP} determined from Section 4.3 of this Handbook	V_{BMP}=		ft^3

Calculate Maximium Depth of the Reservoir Layer

Enter Infiltration rate	I =		in/hr
Enter Factor of Safety, FS (unitless)	FS =		
Obtain from Table 1, Appendix B: "Infiltration Testing" of this BMP Handbook			
	n =	40	%
Calculate D_1. $D_1 = \frac{I\ (in/hr) \times 48\ hrs}{12\ (in/ft) \times (n/100) \times FS}$	D_1 =		ft
Enter depth to historic high groundwater mark (measured from finished grade)			ft
Enter depth to top of bedrock or impermeable layer (measured from finished grade)			ft
D_2 is the smaller of: Depth to groundwater - 11 ft; & Depth to impermeable layer - 6 ft	D_2 =		ft
D_{MAX} is the smaller value of D_1 and D_2, must be less than or equal to 8 fee	D_{MAX} =		ft

Trench Sizing

Enter proposed reservoir layer depth D_R, must be $\leq D_{MAX}$	D_R =		ft
Calculate the design depth of water, d_W			
Design $d_W = (D_R) \times (n/100)$	Design d_W=		ft
Minimum Surface Area, A_S $A_S = \frac{V_{BMP}}{d_W}$	A_S =		ft^2
Proposed Design Surface Area	A_D =		ft^2
Minimum Width = D_R + 1 foot pea gravel			ft
Sediment Control Provided? (Use pulldown)			
Geotechnical report attached? (Use pulldown)			

If the trench has been designed correctly, there should be no error messages on the spreadsheet.

Notes:

Appendix B – Infiltration Testing Guidelines

Infiltration BMPs use the interaction of chemical, physical, and biological processes between soil and water to filter out sediments and constituents from stormwater. Infiltration BMPs require a maximum drawdown time to avoid nuisance issues. Since drawdown time is contingent on the infiltration rate of the underlying soil, tests are used to help establish the vertical infiltration rate of the soil below a proposed infiltration facility. The tests attempt to simulate the physical process that will occur when the facility is in operation.

Section 1 - General Requirements

1.1 - Summary of Requirements

The following is a brief summary of the requirements for all infiltration test reports submitted to the local land use authority[2] for the purpose of water quality BMP design. A checklist form is included at the end of this document.

1. Where infiltration testing is to be performed for establishing the infiltration rate for BMP design, the measured infiltration rate of the underlying soil must be determined using either the single ring infiltrometer test (as described in ASTM D5126, Section 4.1.2.1), the double ring infiltrometer test (ASTM D3385), the well permeameter method (USBR 7300-89), or a percolation test per County of Riverside Department of Environmental Health (RCDEH) test procedures. A general explanation of these test methods can be found in Section 2 of this Appendix. The minimum number of tests required can be found in Table 1 and is dependent upon the type of infiltration test performed.
2. Test pits and borings (ASTM D1452) may be used to determine the USCS series and textural class (SM, CL, etc.) of the soil horizons, the thickness of soil and rock strata, and to estimate the historical high groundwater mark[3]. Test pits or boring logs must be of sufficient depth to establish that a minimum of five feet of permeable soil exists below the infiltration facility, and that there is a minimum of 10 feet between the bottom of the infiltration facility and the historical high groundwater mark[3] (Sections 1.7 and 2.5). The required number of test pits or borings is listed in Table 1.
3. A final report, prepared by a registered civil engineer, geotechnical engineer, certified engineering geologist, or certified hydrogeologist shall be provided to the Local Land Use Authority which demonstrates through infiltration testing and/or soil logs, that the proposed facility location is suitable for the proposed infiltration facility and an infiltration rate shall be recommended. In addition, any requirements associated with impacts to a landslide, erosion or steep slope hazard area should also be addressed in the final report (Section 1.6).

---

[2]*The local land use authority may choose to alter these guidelines and may have different/additional requirements. Consult the local land use authority before conducting tests to determine local requirements.*

[3]*The "historical high groundwater mark" is defined as the groundwater elevation expected due to a normal wet season and shall be obtained by boring logs or test pits.*

4. Tests may be performed only by individuals trained and educated to perform, understand, and evaluate the field conditions. The individual(s) supervising the field work must be named in the final report as described in Item 3 (see Section 1.6).
5. Preliminary site grading plans shall be provided to the local land use authority showing the proposed BMP locations along with section views through each BMP clearly identifying the extents of cut/fill relative to native soil (see Section 1.1).
6. For sites where infiltration BMPs have been determined to be feasible and will be used, infiltration tests shall be performed within the boundaries of the proposed infiltration BMP and at the bottom elevation (infiltration surface) of the proposed infiltration BMP to confirm the suitability of infiltration (see Photo 5).

A Note on "Infiltration Rate" vs. "Percolation Rate"

A common misunderstanding exists that the "percolation rate" obtained from a percolation test is equivalent to the "infiltration rate" obtained from a single or double ring infiltrometer test. While the percolation rate is related to the infiltration rate, percolation rates tend to overestimate infiltration rates and can be off by a factor of 10 or more. However, as is discussed in Section 2.3, the percolation rate can be converted to a reasonable estimate of the infiltration rate using the Porchet Method.

1.2 - Grading Plans

Many projects require a significant amount of grading prior to their construction. It is important to determine if the BMP will be placed in cut or fill since this may affect the performance of the BMP or even the soil. As such, preliminary site grading plans showing the proposed BMP locations are required along with section views through each BMP clearly identifying the extents of cut or fill. In addition, since it is imperative that any testing be performed at the proper elevations and locations, it is highly recommended that the preliminary site grading plans be provided to the engineer/geologist prior to any tests being performed.

1.3 - Cut Condition

Where the proposed infiltration BMP is to be located in a cut condition, the infiltration surface level at the bottom of the BMP might be far below the existing grade. For example, if the infiltration surface of a proposed BMP is to be located at an elevation that is currently beneath 15 feet of cut, how can the proposed infiltration surface be tested?

In order to determine an infiltration rate where the proposed infiltration surface is in a cut condition, the following procedures may be used:

1. The USBR 7300-89, "Procedure for Performing Field Permeability Testing by the Well Permeameter Method" (Section 2.4). Note: the result must be converted to an infiltration rate.
2. The Percolation Test per RCDEH (Section 2.3) may be used. Note: the result must be converted to an infiltration rate.

1.4 - Fill Condition

If the bottom of a BMP (infiltration surface) is in a fill location, the infiltration surface may not exist prior to grading. How then can the infiltration rate be determined? For example, if a proposed infiltration BMP is to be located in 12 feet of fill, how could one reasonably establish an infiltration rate prior to the fill being placed?

Unfortunately, no reliable assumptions can be made about the in-situ properties of fill soil. As such, the bottom, or rather the infiltration surface of the BMP, must extend into natural soil. The natural soil shall be tested at the design elevation prior to the fill being placed.

In some cases, the extension of the BMP down to natural soil may prove infeasible. In that case, another BMP must be selected.

1.5 - Factors of Safety

Long-term monitoring has shown that the performance of working full-scale infiltration facilities may be far lower than the rate measured by small-scale testing. There are several reasons for this:

- Over time, the surface of infiltration facilities can become plugged as sedimentary particles accumulate at the infiltration surface.
- Post-grading compaction of the site can destroy soil structure and seriously impact the facility's performance.
- Soils and soil strata are rarely homogenous, and variations across a site and sometimes even within a BMP footprint, can cause tested infiltration rates to vary widely.
- Testing procedures in general are subject to natural variations and errors which can skew the results.

As such, to obtain an appropriate level of confidence in the final design infiltration rate, factors of safety shall be applied to the tested infiltration rate, I_t, in order to determine the design infiltration rate, I_d. These factors are based on such considerations as the type of tests used, the number of tests performed and whether testing is performed at all. Table 1 provides a complete matrix of testing requirements versus factors of safety.

1.6 - Infiltration Testing Requirements

Table 1 is a list of infiltration BMPs with test regime options and their corresponding design factors of safety. The options are summarized below:

Option 1 - This test regime includes ring infiltrometer type tests, test pit or boring logs and a final report. The minimum required number of tests is as described in Table 1. The minimum required factor of safety for this option is FS=3.

Option 2 - This test regime includes percolation type tests, test pit or boring logs and a final report. The minimum required number of tests is as described in Table 1. The minimum required factor of safety for this option is FS=3.

Option 3 - This test regime includes test pit or boring logs only and a final report. The minimum required number of tests is as described in Table 1. An expected infiltration rate shall be included in the final report based on the specifics of the borings or test pits. The minimum required factor of safety for this option is FS=6. This option may be used for BMPs with a maximum tributary area of five acres. This option is limited to the Preliminary Water Quality Management Plan stage only.

Option 4 - This test regime includes a single test pit or boring log at any representative location on the project site. Plates E-6.1 and E-6.2 of the Riverside County Flood Control and Water Conservation District's (District) Hydrology Manual shall then be used to establish an approximate infiltration rate based on the appropriate Runoff Index and the Antecedent Moisture Content (AMC) as defined on page C-3 of the Hydrology Manual. The minimum required factor of safety for this option is FS=10. This option may be used for BMPs with a maximum tributary area of 5 acres. This option limited to the Preliminary Water Quality Management Plan stage only.

Table 1 - Infiltration Testing Requirements

WQMP Stage	Testing Options	Ring Infiltrometer Tests[1]	Percolation Test[2]	Test Pits or Boring Logs[3]	Final Report[4]	Hydrology Manual[5]	Factor of Safety
Preliminary WQMP	Option 1▶	two tests minimum with at least one per BMP location[6]	-	one boring or test pit per BMP location	Required	-	FS ≥ 3
	Option 2▶	-	four tests min. with at least two per BMP location[6]	one boring or test pit per BMP location	Required	-	FS ≥ 3
	Option 3[7]▶	-	-	one boring or test pit per BMP location	Required	-	FS ≥ 6
	Option 4[7]▶	-	-	one *representative* boring or test pit per site	-	Only	FS ≥ 10
Final WQMP	Option 1▶	two tests minimum with at least one per BMP location[6]	-	one boring or test pit per BMP location	Required	-	FS ≥ 3
	Option 2▶	-	four tests minimum with at least two per BMP location[6]	one boring or test pit per BMP location	Required	-	FS ≥ 3

Table Footnotes:
(1) Ring infiltrometer tests per Section 2.2
(2) Percolation tests per Section 2.3 and well permeameter test per Section 2.4
(3) Test pits or boring logs per Section 2.5
(4) Final Report per Section 1.6
(5) See Plate E-6.2 of the District's Hydrology Manual
(6) For BMPs with a wetted footprint in excess of 10,000 ft^2, provide one (1) ring infiltrometer test or two (2) percolation tests for each additional 10,000 ft^2
(7) This option is limited to BMPs with a tributary drainage area ≤ five acres.

1.7 - Final Report

Where a final report is required, a civil engineer, geotechnical engineer, certified engineering geologist or certified hydrogeologist shall establish whether the location is suitable for the proposed infiltration facility. At least five feet of permeable soil must be present below the infiltration facility and a minimum of 10 feet between the bottom of the infiltration facility and the historical high groundwater mark[3] is required. The signed/stamped report shall include discussion and records of the infiltration testing as well as boring log findings. Based on the results of these tests, the report shall provide an estimate of the infiltration rate found at the location of each proposed infiltration BMP in units of inches per hour. The final report shall present a **recommended design infiltration rate** that includes a factor of safety that is no less than the factor of safety shown in Table 1: the professional preparing and certifying the final report shall recommend a greater safety factor if, based on the professional's judgement, an additional factor of safety is needed to meet project safety and infiltration requirements.

Any requirements associated with impacts to an erosion hazard area, steep slope hazard area, or landslide hazard area should also be addressed in the report. In addition, the report shall include complete field records with the following information:

- Location of the test site.
- Dates of test start and finish.
- Weather conditions, start to finish.
- Names(s) of technician(s).
- Description of test site, including assessment of boring profile and USCS soil classification.
- Depth to the water table and a description of the soils to a depth of at least 10 feet below proposed infiltration surface.
- Type of equipment used to construct the boreholes or test holes (such as backhoe, hollow stem auger, etc.)
- Areas of the rings (if used) or test hole diameter.
- Volume constants for graduated cylinder or Mariotte tube (if used).
- Complete field results in tabular format. Sample test data forms, as well as examples, have been provided following the description of each test in Section 2.
- A plot of the infiltration rate versus total elapsed time. An example is provided following the description of each test in Section 2.
- A labeled keymap showing test and boring locations.
- Confirmation that the soil was pre-saturated in accordance with the testing methods described herein.

Section 2 - Accepted Testing Methods

There is a wide range of different methods for measuring the infiltration rate of a given soil with varying degrees of accuracy and reliability. However, the District will only accept the following test methods:

1. Single Ring Infiltrometer (Per ASTM D5126), Section 2.1.1
2. Double Ring Infiltrometer (Per ASTM D3385), Section 2.1.2
3. Well Permeameter Method (USBR 7300-89), Section 2.4
4. Percolation Test (per County of Riverside Department of Environmental Health procedure), Section 2.3

The following pages of this document provide an overview of these tests. It is recommended that the original standards be referenced.

2.1 - Constant Head vs. Falling Head Method

There are two operational techniques used with all four of the testing techniques herein: the *constant head method,* and the *falling head method*. With the *constant head method*, water is consistently added to both the outer and inner rings (ring infiltrometers) or to the test hole (pecolation test and well permeameter) to maintain a constant level throughout the testing. The volume of water needed to maintain the fixed level of the inner ring is measured. Conversely, in the *falling head method*, the water level is allowed to fall and the time that the water level takes to decrease is measured.

2.2 - Overview of Ring Infiltrometer Test Methods

Ring infiltrometers measure the rate of infiltration at the soil surface. Infiltration is influenced by both saturated hydraulic conductivity, as well as capillary effects. The term *capillary effects* refers to the ability of dry soil to pull, or wick away, water from a zone of saturation faster than would occur if the soil were uniformly saturated. The magnitude of the capillary effect is determined by initial moisture content at the time of testing, the pore size, soil properties (texture, structure) and a number of other factors. The effects of capillarity are short lived and can greatly skew test results. As such, it is critical to obtain steady-state infiltration so that capillary effects are minimized (ASTM 5126).

The ***single ring infiltrometer*** and ***double ring infiltrometer*** methods both employ the use of metal cylinders driven to shallow depths into the test soil. The rings are filled with water and the rate at which the water moves into the soil is measured. This rate becomes constant when the saturated hydraulic conductivity for the particular soil has been reached. This is reflected by the flattening out of the curve generated by sample test data as shown in Figure 2, "Plot of Infiltration Rate vs. Time". While we note that infiltration rate is not exactly the same as saturated hydraulic conductivity, for the purposes of this guidance document they are synonymous.

2.2.1 - Single Ring Infiltrometers

Single ring infiltrometer tests using a ring 40 inches or larger in diameter have been shown to closely match full-scale facility performance (Figures 1 and 2, Photo 1). The cylindrical ring is driven approximately 12 inches into the soil. Water is ponded within the ring above the soil surface. The upper surface of the ring is often covered to prevent evaporation. Using the constant head method, the volumetric rate of water added to the ring, sufficient to maintain a constant head within the ring is measured. The test is complete and the tested infiltration rate, I_t, is determined after the flow rate has stabilized (ASTM D5126).

Photo 1 – Simple Single Ring Infiltrometer

To help maintain a constant head, a variety of devices may be used. A hook gauge, steel tape or rule, length of steel or plastic rod pointed on one end can be used for measuring and controlling the depth of liquid (head) in the infiltrometer ring. If available, a graduated Mariotte tube or automatic flow control system may also be used. Care should be taken when driving the ring into the ground as there can be a poor connection between the ring wall and the soil. This poor connection can cause a leakage of water along the ring wall and an overestimation of the infiltration rate.

The volume of liquid used during each measured time interval may be converted into an incremental infiltration velocity (infiltration rate) using the following equation:

$$I_t = \Delta V/(A*\Delta t)$$

Where:

I_t = tested infiltration rate, in/hr

ΔV = volume of liquid used during time interval to maintain constant head in the ring, in^3

A = internal area of ring, in^2

Δt = time interval, hr.

Final Report - Ultimately, as discussed in Section 1.6, a final report shall be provided and based on the test results, an infiltration rate shall be recommended.

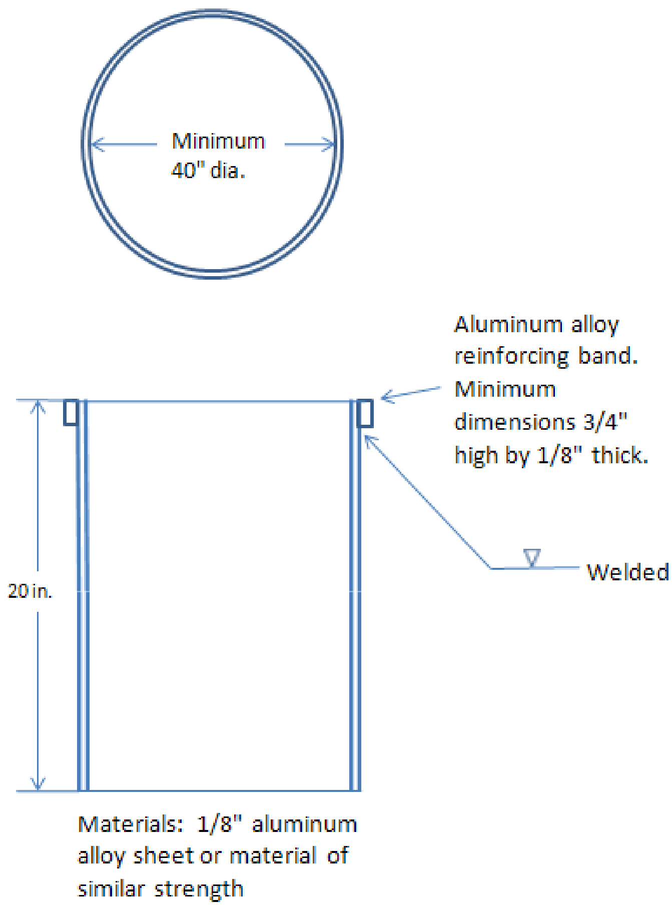


Figure 1- Single Ring Infiltrometer Construction

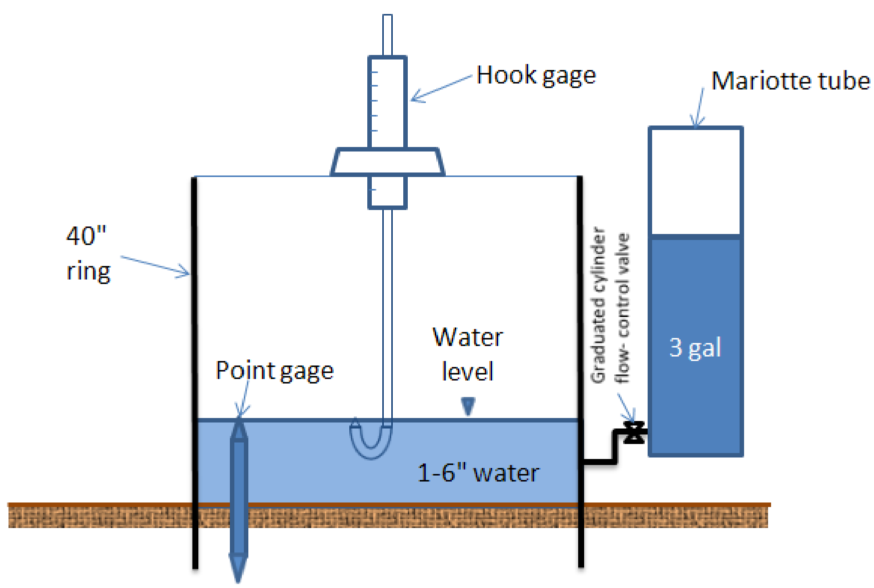


Figure 2- SIngle Ring Setup with Mariotte Tube

SINGLE RING INFILTROMETER TEST DATA					
Project Name and Test Location:		Constants-	Ring Data		Liquid Containers
			Ring Area, A_r (in^2)	Depth of Liquid (in)	Reservoir Container Volume, V_r (in^3/in)
Test By:		USCS Class:		Penetration of Ring into Soil (in.):	
Liquid Used:		pH:		Ground Temp (°F):		at Depth:	
Date of Test:		Depth to Water Table:			
Liquid Level Maintained by using:	( ) Flow Valve ( ) Float Valve ( ) Marriotte Tube ( ) Other:				
Additional Comments:					

Time interval	Time (hr:min)	Dt (min) & Total	Flow Readings: Elev., H (In)	Flow Readings: ΔH (in) & Q_f* (in^3)	Liquid Temp (°F)	Infiltratn Rate, I**(in/hr)	Remarks
1 - Start							
End							
2 - Start							
End							
3 - Start							
End							
4 - Start							
End							
5 - Start							
End							
6 - Start							
End							
7 - Start							
End							
8 - Start							
End							
9 - Start							
End							
10 - Start							
End							
11 - Start							
End							
12 - Start							
End							
13 - Start							
End							
14 - Start							
End							
15 - Start							
End							

*Flow, $Q_f = \Delta H \times V_r$ **Infiltration Rate, $I = (Q_f/A_r)/$

Table 2 – Sample Test Data Form for Single Ring Infiltrometer Test

SINGLE RING INFILTROMETER TEST DATA

Project Name and Test Location:	Constants-	Ring		Liquid Containers
ACME IND. SITE 24166 ELM, RIVERDALE * WESTERN CORNER OF SITE (NEAR WAREHOUSE)		Ring Area, A_r (in^2)	Depth of Liquid (in)	Reservoir Container Volume, V_r (in^3/in)
		1256	4.0	78.54

Test By:	CMD	USCS Class:	SM	Penetration of Ring into Soil (in.):	3.0
Liquid Used:	TAP WATER	pH:	8.0	Ground Temp (°F): 57	at Depth: 16"
Date of Test:	3-21-09	Depth to Water Table:	40 FEET		
Liquid Level Maintained by using:	( ) Flow Valve ( ) Float Valve (X) Marriotte Tube ( ) Other:				
Additional Comments:	DRY GROUND				

Time interval	Time (hr:min)	Dt (min) & Total	Flow Readings: Elev., H (In)	Flow Readings: ΔH (in) & Q_f* (in^3)	Liquid Temp (°F)	Infiltratn Rate, I**(in/hr)	Remarks
1 - Start	10:00	15	3.0	1.45	59	0.36	CLOUDY, SLIGHT WIND
End	10:15	(15)	4.45	114	59		
2 - Start	10:15	15	4.45	2.7	59	0.68	
End	10:30	(30)	7.15	212	59		
3 - Start	10:30	15	7.15	3.35	59	0.84	
End	10:45	(45)	10.5	263	59		
4 - Start	10:45	15	10.5	3.9	59	0.97	
End	11:00	(60)	14.4	306	60		
5 - Start	11:00	30	14.4	9.65	60	1.2	
End	11:30	(90)	24.05	758	61		
6 - Start	11:30	30	24.05	10.8	61	1.4	
End	12:00	(120)	34.85	848	62		
7 - Start	12:10	60	3.5	24.7	62	1.5	REFILLED TUBES
End	13:10	(180)	28.25	1944	63		
8 - Start	13:20	60	2.4	23.9	64	1.5	〃
End	14:20	(240)	26.3	1877	64		
9 - Start	14:30	60	4.3	21.6	64	1.4	〃
End	15:30	(300)	25.9	1696	64		
10 - Start	15:40	60	2.2	20.2	64	1.3	〃
End	16:40	(360)	22.4	1586	64		CLOUDY, SLIGHT WIND
11 - Start							
End							
12 - Start							
End							
13 - Start							
End							
14 - Start							
End							
15 - Start							
End							

*Flow, $Q_f = \Delta H \times V_r$ **Infiltration Rate, $I = (Q_f/A_r)/\Delta t$

Figure 3 – Sample Test Data

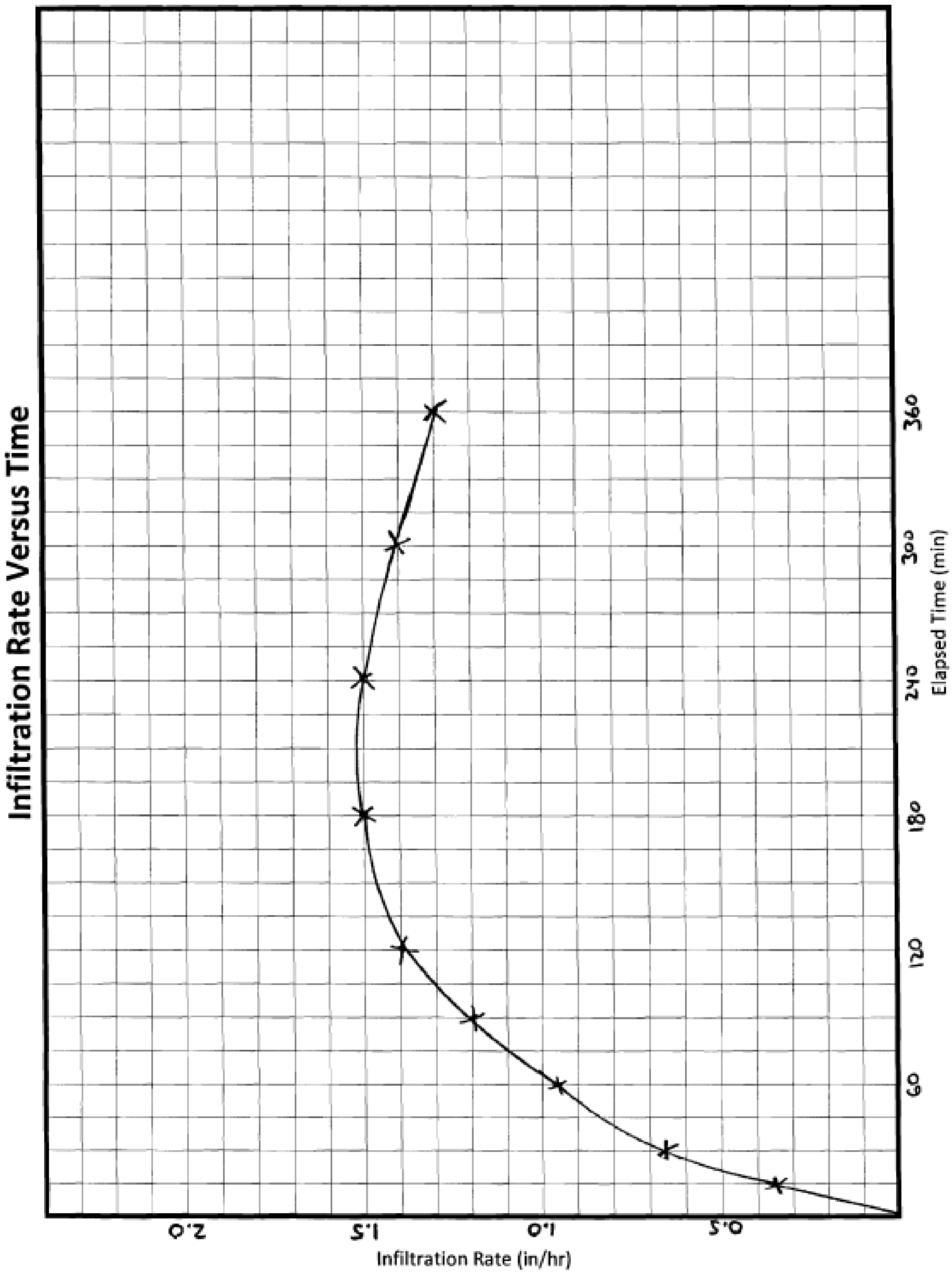


Figure 4 – Plot of Sample Test Data for Single Ring Infiltrometer Test

2.2.2 - Double Ring Infiltrometers

The ***double ring infiltrometer*** test (ASTM D3385) is a well-recognized and documented technique for directly measuring the soil infiltration rate of a site (see Figure 5, 6 and 7; Photos 2, 3, 4 and 5). Double ring infiltrometers were developed in response to the fact that smaller (less than 40 inch diameter) single ring infiltrometers tend to overestimate vertical infiltration rates. This has been attributed to the fact that the flow of water beneath the cylinder is not purely vertical and diverges laterally. Double ring infiltrometers minimize the error associated with the single ring method because the water level in the outer ring forces vertical infiltration of water in the inner ring. Care should be taken when driving the rings into the ground as there can be a poor connection between the ring wall and the soil. This poor connection can cause a leakage of water along the ring wall and an overestimation of the infiltration rate. Another potential source of error is attributed to the size of the cylinders. As such, the use of cylinder sizes less than those prescribed in ASTM D3385 is not recommended.

A typical double ring infiltrometer would consist of a 12 inch inner ring and a 24 inch outer ring. While there are two operational techniques used with the double ring infiltrometer, the constant head method and the falling head method, ASTM D3385 mandates the use of the constant head method. With the constant head method, water is consistently added to both the outer and inner rings to maintain a constant level throughout the testing. The volume of water needed to maintain the fixed level of the inner ring is measured. To help maintain a constant head, a variety of devices may be used. A hook gauge, steel tape or rule, or length of steel or plastic rod pointed on one end, can be used for measuring and controlling the depth of liquid (head) in the infiltrometer ring. If available, a graduated Mariotte tube or automatic flow control system may also be used.

The volume of liquid used during each measured time interval may be converted into an incremental infiltration velocity (infiltration rate) using the following equation:

$$I_t = \Delta V/(A*\Delta t)$$

Where:

I_t = tested infiltration rate, in/hr

ΔV = volume of liquid used during time interval to maintain constant head in the inner ring, in^3

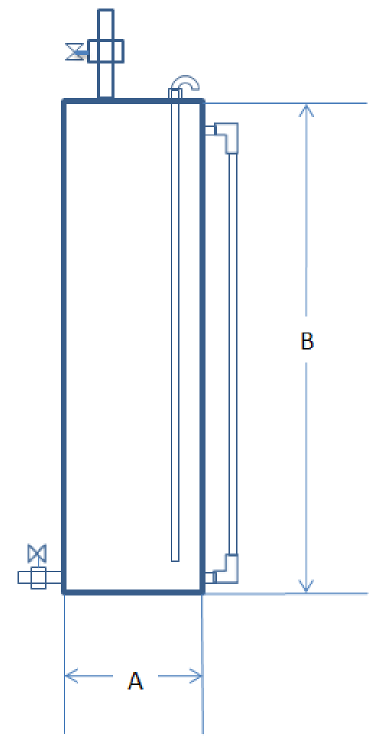
A = area of inner ring, in^2

Δt = time interval, hr.

Final Report - Ultimately, as discussed in Section 1.6, a final report shall be provided, and based on the test results, an infiltration rate shall be recommended.

Photo 2 – Simple Double Ring Infiltrometer

Photo 3 – Pre-fabricated Double Ring Infiltrometer
(Photo courtesy of Turf-Tec International)



Mariotte Tube
Useful Capacity

	1 gal	3 gal
A =	3 in.	6 in.
B =	18 in.	24 in.

Figure 5 - Mariotte Tube

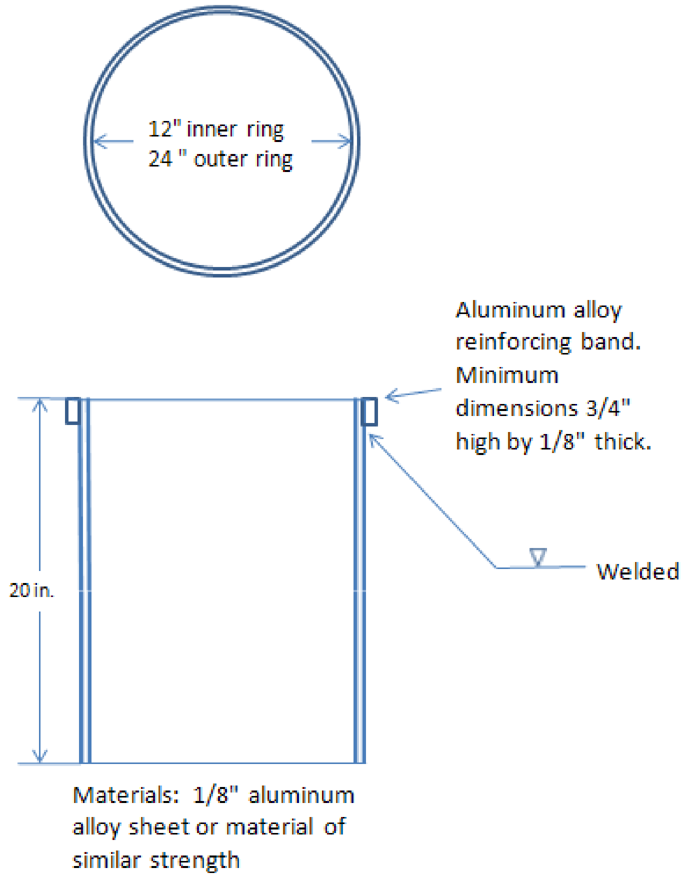


Figure 6- Double Ring Infiltrometer Construction

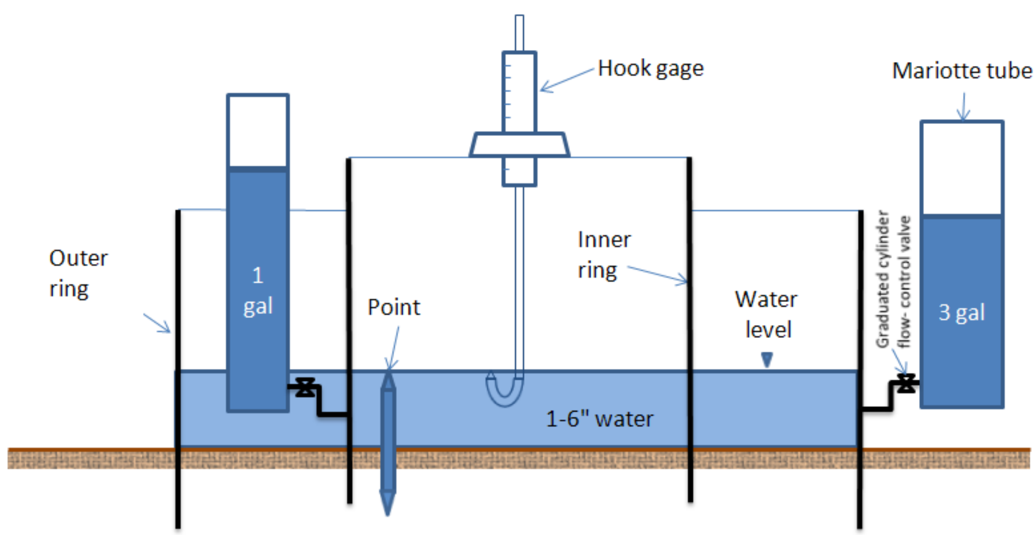


Figure 7- Double Ring Setup with Mariotte Tubes

Photo 4- Double Ring Infiltrometer Set-up with Mariotte Tubes
(Photo courtesy of Turf-Tec International)

Photo 5 – Double Ring Infiltrometer Set-up for Test at Basin Surface Elevation
(Photo courtesy of Turf-Tec International)

DOUBLE RING INFILTROMETER TEST DATA									
Project Name and Test Location:				Constants-		Ring Data		Liquid Containers	
						Area, A_r (in^2)	Depth of Liquid (in)	No.	Vol., V_r (in3/in)
				Inner Ring:					
Test By:		USCS Class:		Annular Space:					
Water Table Depth:		Penetration of Rings into Soil (in.):				Inner:		Outer:	
Date of Test:		Liquid Used:		pH:		Ground Temp (·F):		at Depth:	
Liquid Level Maintained by using:		( ) Flow Valve ( ) Float Valve ( ) Marriotte Tube ( ) Other:							
Additional Comments:									

Time interval	Time (hr:min)	Dt (min) & Total	Inner Ring Elev., H (In)	Inner Ring ΔH (in) &	Annular Ring Elev., H (In)	Annular Ring ΔH (in) &	Liquid Temp °F	Infiltration Rate, I** Inner in/hr	Infiltration Rate, I** Outer in/hr	Remarks
1 - Start										
End										
2 - Start										
End										
3 - Start										
End										
4 - Start										
End										
5 - Start										
End										
6 - Start										
End										
7 - Start										
End										
8 - Start										
End										
9 - Start										
End										
10 - Start										
End										
11 - Start										
End										
12 - Start										
End										
13 - Start										
End										
14 - Start										
End										
15 - Start										
End										

***Flow, Qf = ΔH x Vr** ****Infiltration Rate, I = (Qf/Ar)/Δt**

Table 3 – Sample Test Data Form for Double Ring Infiltrometer Test

DOUBLE RING INFILTROMETER TEST DATA

Project Name and Test Location:	Constants-	Ring Data: Area, A_r (in^2)	Ring Data: Depth of Liquid (in)	Liquid Containers: No.	Liquid Containers: Vol., V_r (in3/in)
ACME Industrial Site 24166 Elm, Riverdale (Western corner of site, near warehouse)	Inner Ring:	113	4	1	78.54
Test By: CMD / USCS Class: SM	Annular Space:	339	4.1	2	176.7

Water Table Depth:	40 ft.	Penetration of Rings into Soil (in.):		Inner:	3.0	Outer:	7.0		
Date of Test:	3/22/09	Liquid Used:	tap water	pH:	8.0	Ground Temp (°F):	57.2	at Depth:	16 in.

Liquid Level Maintained by using: ( ) Flow Valve ( ) Float Valve (X) Marriotte Tube ( ) Other:

Additional Comments: Dry Gound

Time interval	Time (hr:min)	Dt (min) & Total	Inner Ring Elev., H (In)	Inner Ring ΔH (in) &	Annular Ring Elev., H (In)	Annular Ring ΔH (in) &	Liquid Temp °F	Infiltration Rate, I** Inner in/hr	Infiltration Rate, I** Outer in/hr	Remarks
1 - Start	9:00	15	3	0.2	3	0.4	59	0.6	0.8	Cloudy, slight wind
End	9:15	15	3.2	15.71	3.4	70.68	59			
2 - Start	9:15	15	3.2	0.35	3.4	0.6	59	1.0	1.3	
End	9:30	30	3.55	27.49	4	106	59			
3 - Start	9:30	15	3.55	0.5	4	0.9	59	1.4	1.9	
End	9:45	45	4.05	39.27	4.9	159	59			
4 - Start	9:45	15	4.05	0.65	4.9	1.2	59	1.8	2.5	
End	10:00	60	4.7	51.05	6.1	212	60			
5 - Start	10:00	30	4.7	1.5	6.1	2.65	60	2.1	2.8	
End	10:30	90	6.2	117.8	8.75	468.3	61			
6 - Start	10:30	30	6.2	1.7	8.75	2.75	61	2.4	2.9	
End	11:00	120	7.9	133.5	11.5	485.9	62			
7 - Start	11:10	60	3.25	3.75	2.5	5.9	62	2.6	3.1	
End	12:10	180	7	294.5	8.4	1043	63			Refilled tubes
8 - Start	12:20	60	3.5	3.9	3	5.7	64	2.7	3.0	
End	13:20	240	7.4	306.3	8.7	1007	64			Refilled tubes
9 - Start	13:30	60	3	3.6	3.1	5.5	64	2.5	2.9	
End	14:30	300	6.6	282.7	8.6	971.9	64			Refilled tubes
10 - Start	14:40	60	3.25	3.45	3	5.4	64	2.4	2.8	
End	15:40	360	6.7	271	8.4	954.2	64			Refilled tubes
11 - Start	15:50	60	3.3	3.4	2.9	5	64	2.4	2.6	
End	16:50	420	6.7	267	7.9	883.5	64			Refilled tubes
12 - Start	18:00	60	3	3.5	3.1	4.9	64	2.4	2.6	Refilled tubes
End	19:00	480	6.5	274.9	8	865.8	64			Cloudy, no wind
13 - Start										
End										
14 - Start										
End										
15 - Start										
End										

*Flow, $Qf = \Delta H \times Vr$ **Infiltration Rate, $I = (Qf/Ar)/\Delta t$

Table 4 – Sample Test Data Form for Double Ring Infiltrometer Test

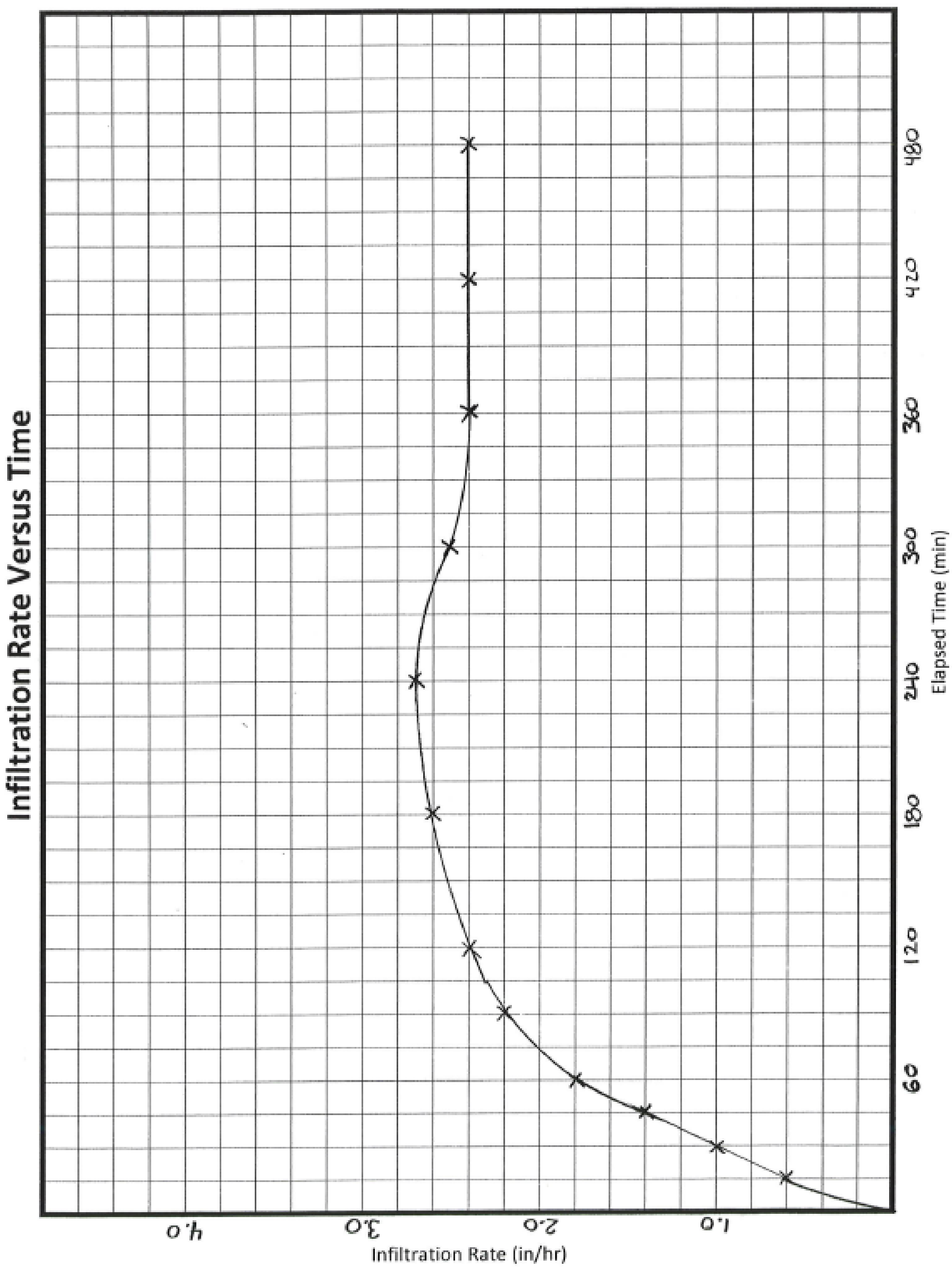


Figure 8 – Plot of Sample Test Data for Double Ring Infiltrometer Test

2.3 - Percolation Tests

The ***percolation test*** is widely used for assessing the suitability of a soil for onsite wastewater disposal. Depending on the required depth of testing, there are two versions of the percolation test. For shallow depth testing (less than 10 feet), the procedure would be as shown in Figure 8 (Photo 6). For deep testing (10 feet to 40 feet), the procedure is as shown in Figure 9. For deep testing, special care must be taken to ensure that caving of the sidewalls does not occur.

This test measures the length of time required for a quantity of water to infiltrate into the soil and is often called a "percolation rate". It should be noted that the percolation rate is related to, but not equal to, the infiltration rate. While an infiltration rate is a measure of the speed at which water progresses downward into the soil, the percolation rate measures not only the downward progression but the lateral progression through the soil as well. This reflects the fact that the surface area for infiltration testing would include only the horizontal surface while the percolation test includes both the bottom surface area and the sidewalls of the test hole. However, there is a relationship between the values obtained by a percolation test and infiltration rate. Based on the "Porchet Method"[4], the following equation may be used to convert percolation rates to the tested infiltration rate, I_t:

$$I_t = \frac{\Delta H \, \pi r^2 \, 60}{\Delta t(\pi r^2 + 2\pi r H_{avg})} = \frac{\Delta H \, 60 \, r}{\Delta t(r + 2H_{avg})}$$

Where:

I_t = tested infiltration rate, inches/hour
ΔH = change in head over the time interval, inches
Δt = time interval, minutes
$^*r$ = effective radius of test hole
H_{avg} = average head over the time interval, inches

An example of this procedure is provided on page 26 based data form Table 5, *Sample Percolation Test Data*. Figure 11 provides a plot of the converted percolation test data.

*Where a rectangular test hole is used, an equivalent radius should be determined based on the actual area of the rectangular test hole. (i.e., $r = (A/\pi)^{0..5}$)

Note to the designer: The values obtained using this method may vary from those obtained from methods considered to be more accurate. The designer is encouraged to explore the derivation of these equations (Ritzema; Smedema)

Final Report - Ultimately, as discussed in Section 1.6, a final report shall be provided and, based on the test results, an infiltration rate shall be recommended.

[4] *H.P. Ritzema, "Drainage Principles and Applications," International Institute for Land Reclamation and Improvement (ILRI), Publication 16, 2nd revised edition, 1994, Wageningen, The Netherlands.*

Percolation Test Procedure

Only those individuals trained and educated to perform, understand and evaluate the field conditions and tests may perform these tests. This would include those who hold one of the following State of California credentials and registrations: Professional Civil and Geotechnical Engineers, Certified Engineering Geologist and Certified Hyrdrogeologist. The District will only approve the percolation test method described in this Section.

When the percolation testing has been completed, a 3 foot long surveyor's stake (lath) shall be flagged with highly visible banner tape and placed in the location of the test indicating date, test hole number as shown on the field data sheet, and firm performing the test. Field data shall be included in the Final Report as described in Section 1.6.

Shallow Percolation Test (less than 10 feet)

Test Preparation

1) The test hole opening shall be between eight and 12 inches in diameter or between seven and 11 inches on each side if square.
2) The bottom elevation of the test hole shall correspond to the bottom elevation of the proposed basin (infiltration surface). Keep in mind that this procedure will require the test hole to be filled with water to a depth of at least five times the hole's radius.
3) The bottom of the test hole shall be covered with two inches of gravel.
4) The sides of the hole shall remain undisturbed (not smeared) after drilling and any cobbles encountered left in place.
5) **Pre-soaking** shall be used with this procedure. Invert a full five gallon bottle (more if necessary) of clear water supported over the hole so that the water flow into the hole holds constant at a level at least five times the hole's radius above the gravel at the bottom of the hole. Testing may commence after all of the water has percolated through the test hole or after 15 hours has elapsed since initiating the pre-soak. However, to assure saturated conditions, testing must commence no later than 26 hours after all pre-soak water has percolated through the test hole. The use of the "continuous pre-soak procedure" is no longer accepted. When sandy soils (as described below) are present, the test shall be run immediately.

Test Procedure

Test hole shall be carefully filled with water to a depth equal to at least five times the hole's radius (H/r>5) above the gravel at the bottom of the test hole prior to each test interval.

- In **sandy soils**, when two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Measurements shall be taken with a precision of 0.25 inches or better. The drop that occurs during the final 10 minutes is used to calculate the percolation rate. Field data must show the two 25 minute readings and the six 10 minute readings.

- In **non-sandy soils**, obtain at least 12 measurements per hole over at least six hours with a precision of 0.25 inches or better. From a fixed reference point, measure the drop in water level over a 30 minute period for at least six hours, refilling after every 30 minute reading. The total depth of the hole must be measured at every reading to verify that collapse of the borehole has not occurred. The drop that occurs during the final reading is used to calculate the percolation rate.

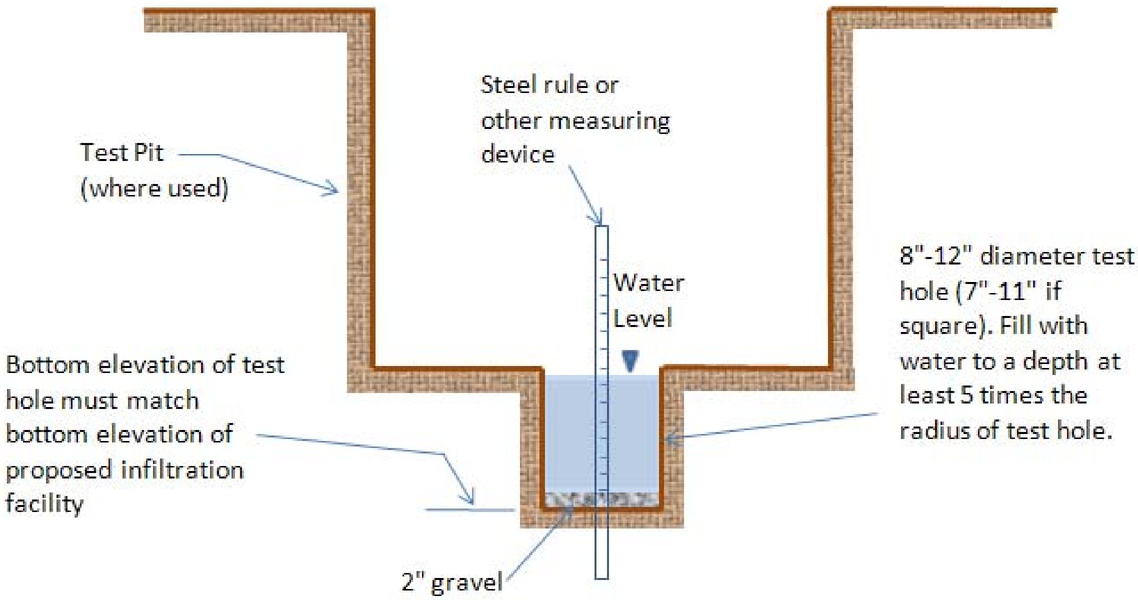


Figure 9- Test Pit for Shallow Percolation Test

Deep Percolation Test (Depths 10-40 feet)

Test Preparation

1) Borehole diameter shall be either six inch or eight inch only. No other diameter test holes will be accepted.
2) The bottom elevation of the test hole shall correspond to the bottom elevation of the proposed basin (infiltration surface). Keep in mind that this procedure will require the test hole to be filled with water to a depth of at least five times the hole's radius.
3) The bottom of the test hole shall be covered with two inches of gravel.
4) The sides of the hole shall remain undisturbed (not smeared) after drilling and any cobbles encountered left in place. Special care should be taken to avoid cave-in.
5) **Pre-soaking** shall be used with this procedure. Invert a full five gallon bottle of clear water supported over the hole so that the water flow into the hole holds constant at a maximum depth of four feet below the surface of the ground or if grading cuts are anticipated, to the approximate elevation of the **top** of the basin but at least five times the hole's radius (H/r>5). Pre-soaking shall be performed for 24 hours unless the site consists of sandy soils containing little or no clay. If sandy soils exist as described below, the tests may then be run after a two hour pre-soak. However, to assure saturated conditions, testing must commence no later than

26 hours after all pre-soak water has percolated through the test hole. The use of the "continuous pre-soak procedure" is no longer accepted. When sandy soils (as described below) are present, the test shall be run immediately.

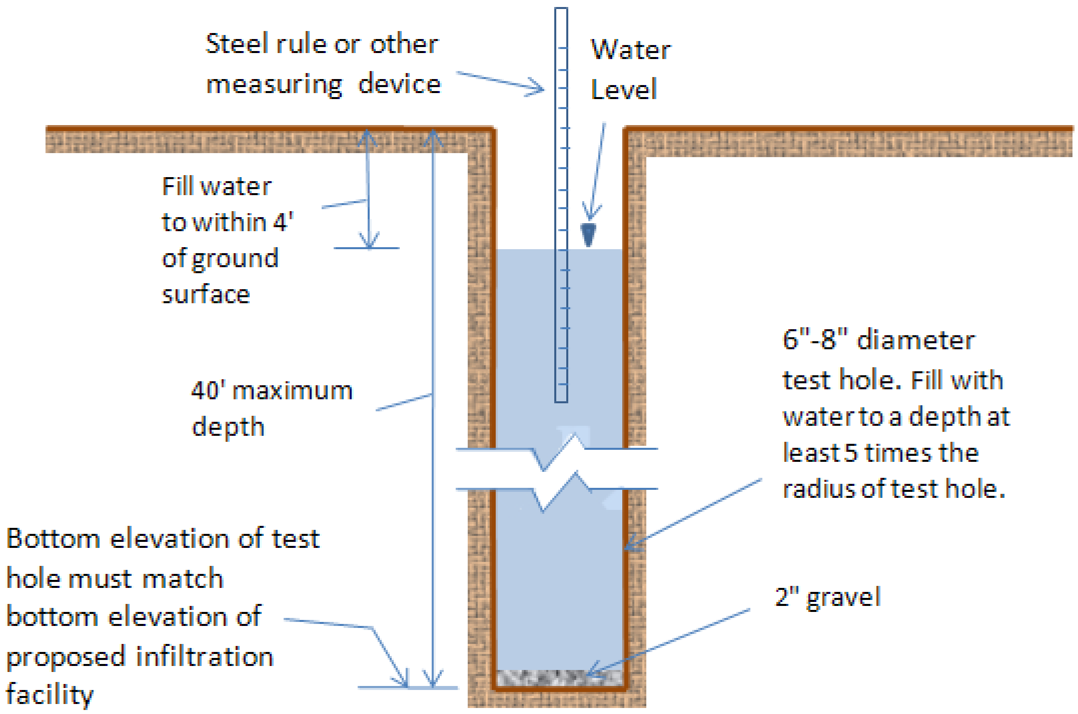


Figure 10- Test Pit for Deep Percolation Test

Test Procedure

Carefully fill the hole with clear water to a maximum depth of four feet below the surface of the ground or, if grading cuts are anticipated, to the approximate elevation of the **top** of the basin. However, at a minimum, the bore hole shall be filled with water to a depth equal to five times the hole's radius (H/r>5).

- In **sandy soils**, when two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Measurements shall be taken with a precision of 0.25 inches or better. The drop that occurs during the final 10 minutes is used to calculate the percolation rate. Field data must show the two 25 minute readings and the six 10 minute readings.
- In **non-sandy soils**, the percolation rate measurement shall be made on the day following initiation of the pre-soak as described in Item above. From a fixed reference point, measure the drop in water level over a 30 minute period for at least 6 hours, refilling after every 30 minute reading. Measurements shall be taken with a precision of 0.25 inches or better. The total depth of hole must be measured at every reading to verify that collapse of the borehole has not occurred. The drop that occurs during the final reading is used to calculate the percolation rate.

Photo 6 – Percolation Test Pit. Use of perforated PVC pipe is a variation.

Percolation Test Data Sheet

Project:			Project No:			Date:	
Test Hole No:			Tested By:				
Depth of Test Hole, D_T:			USCS Soil Classification:				
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round)=			Sides (if rectangular)=				
Sandy Soil Criteria Test*							
Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1							
2							

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1							
2							
3							
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							

COMMENTS:

Table 5 – Sample Test Data Form for Percolation Test

Percolation Rate Conversion

Example:

The bottom of a proposed infiltration basin would be at five feet below natural grade. Percolation tests are performed within the boundaries of the proposed basin location with the depth of the test hole set at the infiltration surface level (bottom of the basin). The Percolation Test Data Sheet (Table 5) is prepared as the test is being performed. After the minimum required number of testing intervals, the test is complete. **The data collected at the final interval is as follows:**

Time interval, Δt = 10 minutes
Initial Depth to Water, D_0 = 12.25 inches
Final Depth to Water, D_f = 13.75 inches
Total Depth of Test Hole, D_T = 60 inches
Test Hole Radius, r = 4 inches

The conversion equation is used:

$$I_t = \frac{\Delta H\ 60\ r}{\Delta t(r+2H_{avg})}$$

"H_o" is the initial height of water at the selected time interval.

$$H_o = D_T - D_0 = 60 – 12.25 = \underline{\textbf{47.75 inches}}$$

"H_f" is the final height of water at the selected time interval.

$$H_f = D_T - D_0 = 60 - 13.75 = \underline{\textbf{46.25 inches}}$$

"ΔH" is the change in height over the time interval.

$$\Delta H = \Delta D = H_o - H_f = 47.75 – 46.25 = \underline{\textbf{1.5 inches}}$$

"H_{avg}" is the average head height over the time interval.

$$H_{avg} = (H_o - H_f)/2 = (47.75 – 46.25)/2 = \underline{\textbf{47.0 inches}}$$

"I_t" is the tested infiltration rate.

$$I_t = \frac{\Delta H\ 60\ r}{\Delta t(r+2H_{avg})} = \frac{(1.5\text{ in})(60\text{ min/hr})(4\text{ in})}{(10\text{ min})((4\text{ in}) + 2(47\text{ in}))} = \underline{\textbf{0.37 in/hr.}}$$

Percolation Test Data Sheet

Project:	ACME SITE	Project No:	1106 B	Date:	2-18-09
Test Hole No:	3	Tested By:	CMD		
Depth of Test Hole, D_T:	60 IN.	USCS Soil Classification:	SM		

Test Hole Dimensions (inches)			Length	Width
Diameter (if round)=	8	Sides (if rectangular)=		

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	8:00	8:25	25	12.0	19.5	7.5	Y
2	8:30	8:55	25	12.0	18.75	6.75	Y

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	9:00	9:10	10	12.0	14.25	2.25	4.4
2	9:10	9:20	10	11.5	13.5	2.0	5.0
3	9:20	9:30	10	12.0	14.0	2.0	5.0
4	9:30	9:40	10	11.75	13.5	1.75	5.7
5	9:40	9:50	10	12.0	13.5	1.5	6.7
6	9:50	10:00	10	12.25	13.75	1.5	6.7
7							
8							
9							
10							
11							
12							
13							
14							
15							

Data used for conversion to Infiltration rate.

COMMENTS: OVERCAST (62°F). GROUND DRY. FIRST (2) MEASUREMENTS MET SANDY SOIL CRITERIA.

Table 6 – Sample Percolation Test Data

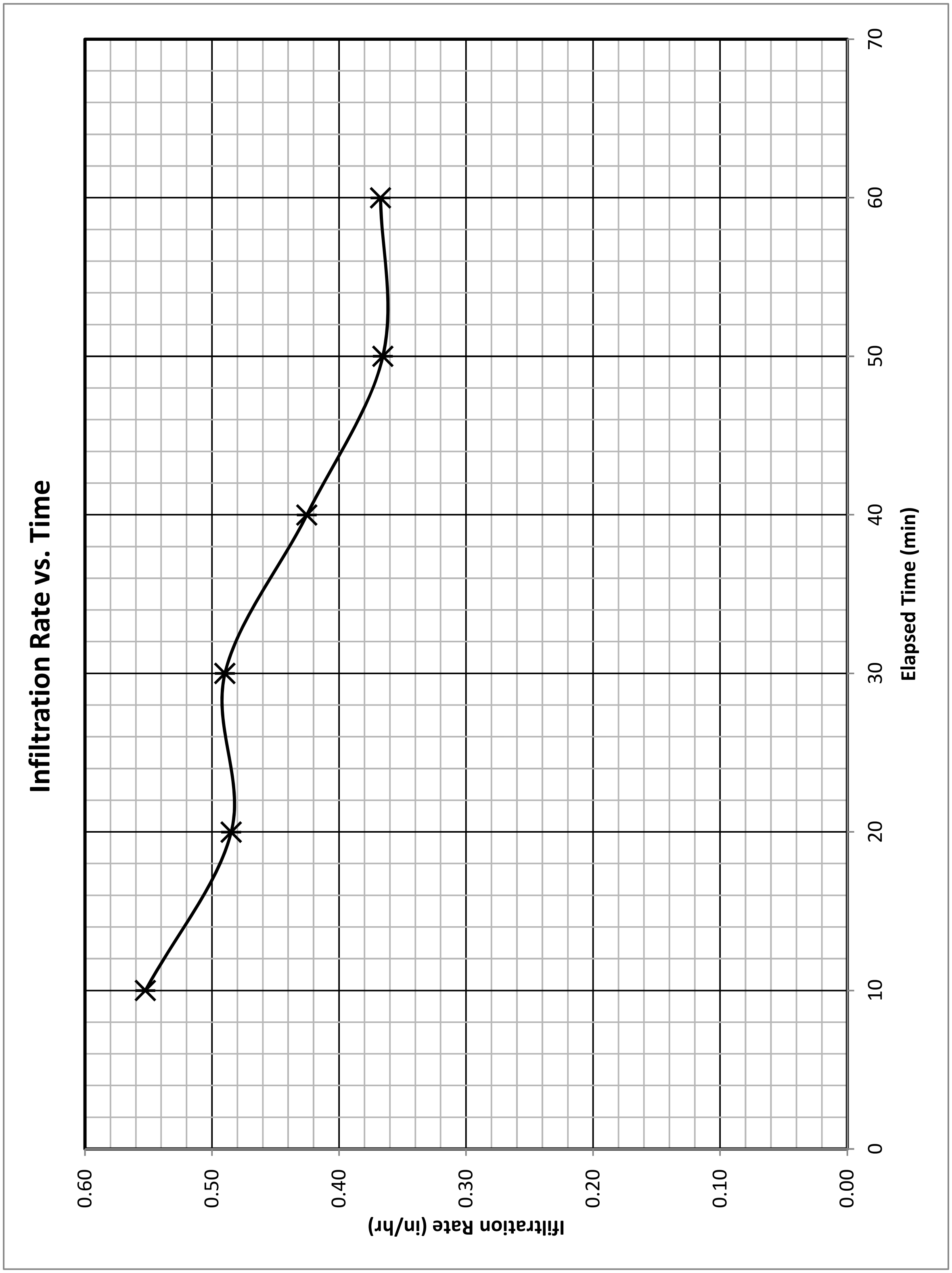


Figure 11 – Plot of Converted Percolation Test Data

2.4 - Field Permeability Test (Well Permeameter Method USBR 7300-89)

Similar to a constant-head version of the percolation test used for seepage pit design is the Well Permeameter Method of the United States Bureau of Reclamation. USBR 7300-89[5] is an in-hole hydraulic conductivity test performed by drilling test wells with a six to eight inch diameter auger to the desired depth. This test measures the rate at which water flows into the soil under constant-head flow conditions and is used to determine field-saturated hydraulic conductivity. As with the percolation test, the rate determined with this test is a "percolation rate" and is related, but not equal, to the infiltration rate. Infiltration rate is a measure of the speed at which water progresses downward into the soil. A percolation rate measures not only the downward progression, but the lateral progression through the soil. However, this procedure uses the following equation(s) to establish an infiltration rate:

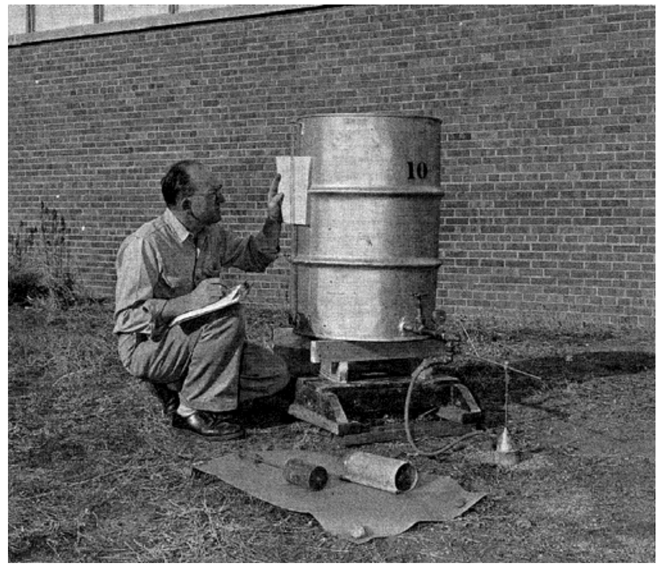


Photo 7 - Typical Well Permeamater Test Installation

Condition I: Typical condition (see Figure 12). The distance between the historical high water mark[6] and the water surface in the well is at least three (3) times the height of water in the well. In addition, there must be at least 10 feet from the bottom of the well to the historical high water table and at least five feet to impervious strata.

$$K_s = \frac{Q(\mu_T/\mu_{20})}{2\pi H^2}\left[\ln\left[\frac{H}{r}+\sqrt{\left(\frac{H}{r}\right)^2+1}\right]-\frac{\sqrt{1+\left(\frac{H}{r}\right)^2}}{\frac{H}{r}}+\frac{r}{H}\right]$$

Where:

K_s = saturated hydraulic conductivity (infiltration rate, inches/hour)
H = height of water in well (inches)
Q = percolation flow rate from selected time interval (cubic inches/hour)
r = effective radius of well (inches)
μ_T = viscosity of water at test temperature, T
μ_{20} = viscosity of water at 20°C

[5] *A detailed description of this procedure along with a complete example using the associated equations can be found in the United States Bureau of Mines and Reclamation (USBR) document 7300-89.*

[6] *The "historical high groundwater mark" is defined as the geoundwater elevation expected due to a normal wet season, and shall be obtained by boring logs or test pits.*

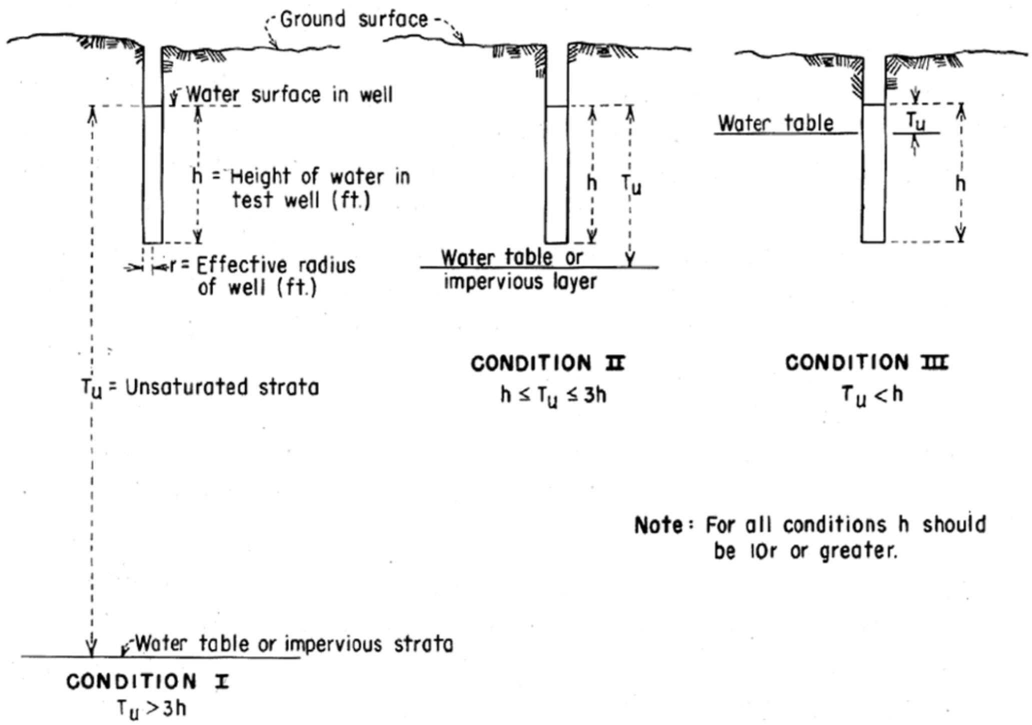


Figure 12 – Site Conditions Govern Procedure to be Used

Condition II: The distance between the historical high groundwater mark[6] and the water surface in the well is less than three (3) times, but at least equal to, the height of water in the well. In addition, there must be at least 10 feet from the bottom of the well to the historical high water mark[6] and at least 5 feet to impervious strata.

$$K_s = \frac{Q(\mu_{20}/\mu_T)}{2\pi H^2}\left[\frac{\ln\left(\frac{H}{r}\right)}{\frac{1}{6}+\frac{1}{3}\left(\frac{H}{T_u}\right)^{-1}}\right]$$

Where:

K_s = saturated hydraulic conductivity (infiltration rate, inches/hour)
H = height of water in well (inches)
Q = percolation flow rate from selected time interval (cubic inches/hour)
r = effective radius of well (inches)
μ_T = viscosity of water at water temperature, T
μ_{20} = viscosity of water at 20° C
T_u = unsaturated distance between the water surface and the water table or impervious strata

Condition III: Unacceptable location. The distance between the historical high groundwater mark[6] and the water surface in the well is less than the height of water in well. As such, the base of the BMP would not be 10 feet above the historical high water mark[6] or 5 feet from impervious strata.

Final Report - Ultimately, as discussed in Section 1.6, a final report shall be provided and, based on the test results, an infiltration rate shall be recommended.

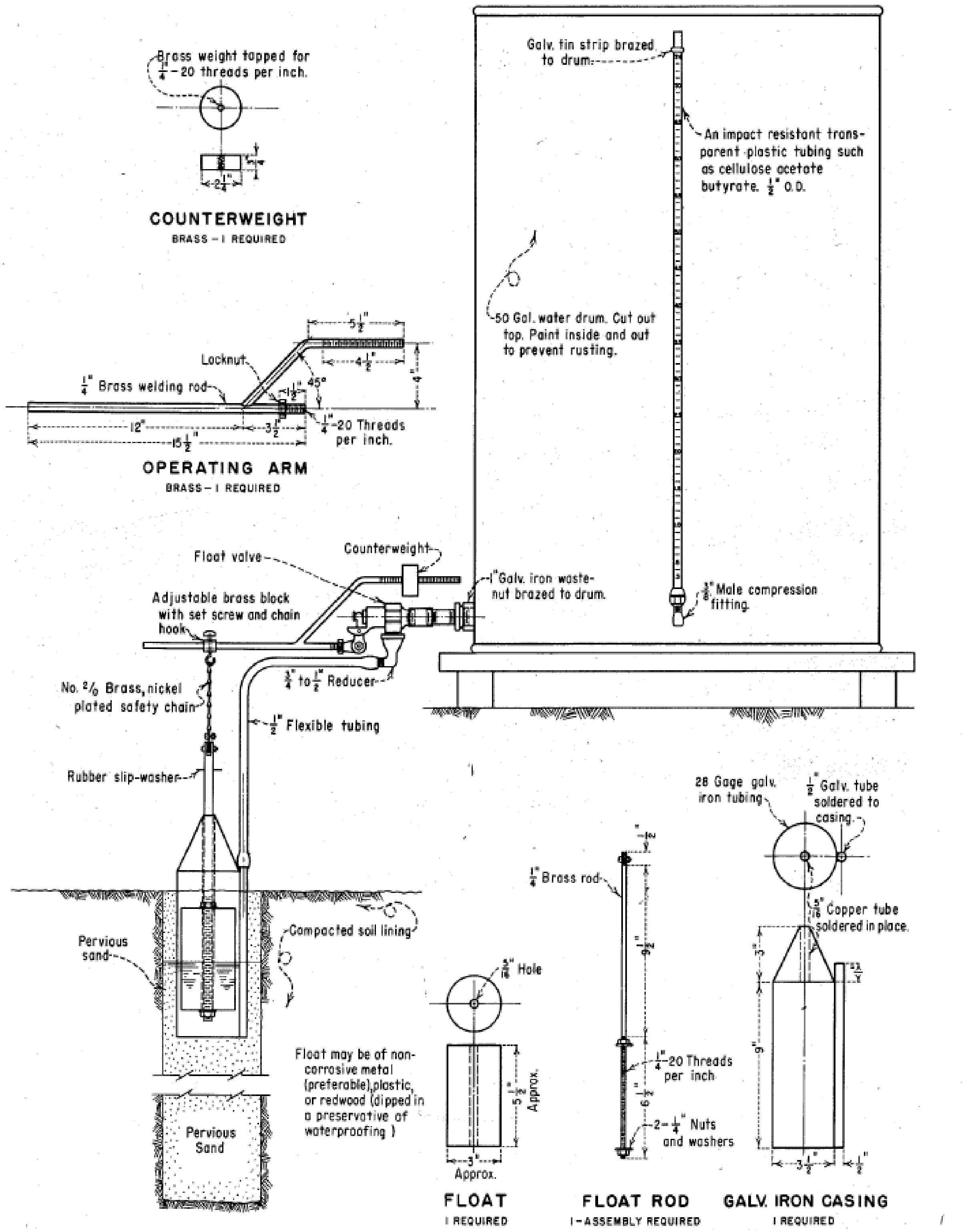


Figure 11 – Well Permeameter Test Equipment

2.5 - Borings and Test Pits

Borings and test pits are used to determine the thickness of soil and rock strata, estimate the depth to groundwater, obtain soil or rock specimens and perform field tests such as standard penetration tests (SPTs) or cone penetration tests (CPTs).

Test pits and trenches may be used to evaluate near-surface conditions up to about 15 feet deep but are often used for performing subsurface exploration at shallower depths. Test pits are often square in plan view and may be dug with shovels in less accessible areas. Trenches are long and narrow excavations usually made by a backhoe or bulldozer.

Borings (ASTM D1452) are generally used to investigate deeper subsurface conditions. A cylindrical hole is drilled into the ground for the purpose of investigating subsurface conditions, performing field tests, and obtaining soil, rock, or underground specimens for testing. Borings can be excavated by hand (e.g., hand auger), although the usual procedure is to use mechanical equipment to excavate the borings.

Whatever method is used, testing shall be sufficient to establish USCS series and textural class (SM, CL, etc) of the soil beneath the infiltration surface of the BMP and of sufficient depth to establish that a minimum of five feet of permeable soil exists below the infiltration facility and that there is a minimum of 10 feet between the bottom of the infiltration facility and the historical high groundwater mark[6].

Photo 8- Auger Boring Rig

Photo 9 – Test Pit Excavation

Infiltration Test Requirement Checklist

___ Where infiltration testing is to be performed, the measured infiltration rate of the underlying soil must be determined using either the single ring infiltrometer test (as described in ASTM D5126, Section 4.1.2.1), the double ring infiltrometer test (ASTM D3385), the well permeameter method (USBR 7300-89), or a percolation test per County of Riverside Department of Environmental Health (RCDEH) test procedures. A general explanation of these test methods can be found in Section 2 of this Appendix. The minimum number of tests required can be found in Table 1 and is dependent upon the type of infiltration test performed.

___ Test pits and borings (ASTM D1452) may be used to determine the USCS series and textural class (SM, CL, etc.) of the soil horizons throughout the depth of boring log or pit, the thickness of soil and rock strata, and estimate the historical groundwater depth. Test pits or boring logs must be of sufficient depth to establish that a minimum of five feet of permeable soil exists below the infiltration facility and that there is a minimum of 10 feet between the bottom of the infiltration facility and the historical high groundwater mark[6] (Section 1.6 and 2.5). The required number of test pits or borings is listed in Table 1.

___ A final report, prepared by a registered civil engineer, geotechnical engineer, certified engineering geologist or certified hydrogeologist shall be provided to the local land use authority which demonstrates through infiltration testing and/or soil logs that the proposed facility location is suitable for the proposed infiltration facility and an infiltration rate shall be recommended. In addition, any requirements associated with impacts to a landslide, erosion or steep slope hazard area should also be addressed in the final report (Section 1.6).

___ Tests may be performed only by individuals trained and educated to perform, understand and evaluate the field conditions. The individual(s) supervising this field work must be named along with their education or training background in the final report as described in Item 3 (Section 1.6).

___ Preliminary site grading plans shall be provided to the local land use authority showing the proposed BMP locations along with section views through each BMP clearly identifying the extents of cut/fill.

___ All infiltration tests shall be performed within the boundaries of the proposed infiltration BMP and at the bottom elevation (infiltration surface) of the proposed infiltration facility (see Photo 5).

This page intentionally left blank

to facilitate double-sided printing

Appendix C – Underdrain Guidelines

Where underdrains are specified, the following information provides guidance for underdrain requirements.

Underdrain Material Types

Underdrain pipe shall be sixch diameter ABS pipe or PVC pipe. ABS pipe shall meet the requirements of ASTM Designation D-2751, SDR 23.5, and PVC pipe shall meet the requirements of ASTM Designation D-2665. Perforations shall be as described in ASTM Designation C-700. It should be noted that placing the pipe such that the perforations are oriented upward may help to maximize infiltration in unlined BMPs with underdrains. If the BMP is constructed with an impermeable liner, the perforations should be angled downward to maximize the volume of water that will be drained from the BMP.

Underdrain Connections

Pipe joints and storm drain structure connections must be adequately sealed to avoid piping conditions (water seeping through pipe or structure joints). Pipe sections shall be coupled using suitable connection rings and flanges. Field connections to storm drain structures and pipes shall be sealed with polymer grout material that is capable of adhering to surfaces. Underdrain pipe shall be capped (at structure) until completion of site construction. Underdrains connected directly to a storm drainage structure shall be non-perforated for an appropriate distance from the structure interface to avoid possible piping problems.

Underdrain Slope

Underdrains must "daylight" or connect to an existing drainage system to achieve positive flow. All underdrains must be placed with a minimum slope of 0.5% (s = 0.005 ft/ft).

Underdrains Layout and Spacing

Typically, there are two main layouts for underdrains. One is a non-perforated central collector pipe with perforated lateral feeder pipes; the other is simply a series of longitudinal perforated pipes. Both layouts connect to a non-perforated outlet pipe before “daylighting" or connecting to an existing drainage system. The minimum spacing is shown below.

BMP Type	Underdrains Center to Center Spacing
Sand Filter Basin	20’
Extended Detention Basins (Bottom stage ≥ 500 sq ft.)	20’
Extended Detention Basins (Bottom stage < 500 sq ft.)	10’
Bioretention Facility	5’

Underdrain Gravel

Gravel bed materials should be used to protect an underdrain pipe and to reduce clogging potential. Placement of gravel over the underdrain must be done with care. Avoid dropping the gravel from excessive heights from a backhoe or front-end loader bucket. Spill directly over underdrain and spread manually.

Recommended construction specifications for gravel used to protect underdrains are as follows:

- AASHTO #57 stone preferred
- Geotextile fabrics should be avoided because tearing and/or plugging can dramatically affect performance. If the designer is concerned about the engineered soil media migrating into the underdrain, a 3 inch thick layer of "pea gravel" may be added to create a “choker” course.

Maintenance

Access for cleaning underdrains is required for each system. Cleanouts, with diameters equal to the underdrain, should extend six inches above the media and have a lockable screw cap for easy access. Cleanouts should be located for every 50 feet of lateral, at the collector drain line connection, and at any bends.

Underdrain Orifice Plate

When designing a BMP to meet HCOC criteria in addition to water quality criteria, it is sometimes necessary to install an orifice plate near the downstream end of the underdrain system. The orifice plate restricts the opening of the underdrain to mitigate flows to a specific lower flow threshold. Proper maintenance access should be provided to the orifice plate location to facilitate maintenance activities, specifically the removal of accumulated sediment and debris upstream of the orifice plate.

Class V Well Status

Section 3.5 of this Handbook discusses BMP design features that may result in the BMP being classified as a Class V injection well. Underdrains, among other features, may trigger this classification. Refer to Section 3.5 of this Handbook for guidance on Class V wells.