Riverside County

Santa Margarita River Watershed Region

Design Handbook

for

Low Impact Development

Best Management Practices

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Riverside County - Santa Margarita River Watershed Region

Design Handbook for Low Impact Development Best Management Practices

1.0 Introduction

What is Low Impact Development?

According to the State Water Resources Control Board, Low Impact Development (LID) is:

> *… a sustainable practice that benefits water supply and contributes to water quality protection. Unlike traditional storm water management, which collects and conveys storm water runoff through storm drains, pipes, or other conveyances to a centralized storm water facility, LID takes a different approach by using site design and storm water management to maintain the site's pre-development runoff rates and volumes. The goal of LID is to mimic a site's predevelopment hydrology by using design techniques that infiltrate, filter, store, evaporate, and detain runoff close to the source of rainfall.*[1]

When implemented correctly on a site, LID provides two primary benefits: 1) The post-construction site hydrology will more closely mimic the pre-development hydrology, thus reducing the downstream erosion that may occur due to increased runoff from impervious surfaces; and 2) Pollutants in runoff from the site will be significantly reduced.

Additionally, the California Stormwater Quality Association (CASQA) LID Manual[2] identifies that a properly and effectively designed site will incorporate two forms of LID: LID Principles and LID BMPs. Whereas LID Principles focus on planning and designing a site in a manner that minimizes the causes, or drivers, of project impacts (sometimes referred to as site design), this Handbook discusses LID BMPs which are implemented to help mitigate any impacts that are otherwise unavoidable.

Notes on terminology: This Handbook uses the term "LID Principles" or "Site Design" to refer to BMPs described in Provision E.3.a(3) of the Regional MS4 Permit (note the Permit calls these BMPs "Low Impact Development"). This Handbook uses the terms "LID BMPs" and "Treatment

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[1] State Water Resources Control Board, *Low Impact Development – Sustainable Storm Water Management*, 2010; http://www.waterboards.ca.gov/water_issues/programs/low_impact_development/index.shtml

[2] California Stormwater Quality Association, *Low Impact Development Manual for Southern California: Technical Guidance and Site Planning Strategies*, April 2010

Control BMPs" are used to refer to classes of Structural Stormwater Pollutant Control BMPs described in Provision E.3.c(1) of the Regional MS4 Permit.

About this Handbook

This Handbook supplements the Water Quality Management Plan (WQMP) for the Santa Margarita River Watershed Region (SMR) of Riverside County by providing guidance for the planning, design and maintenance of Low Impact Development (LID) BMPs (i.e. Permit Provision E.3.c(1))which may be used to mitigate the water quality impacts of development within the SMR Watershed.

This Handbook contains detailed information and designs for twelve (12) LID BMPs that are designed to encourage replication of the site's natural hydrologic processes. This includes maximizing direct or incidental infiltration and evapotranspiration, and using vegetation and other biological processes to filter and absorb pollutants. For each BMP, pertinent information is provided such as the maximum tributary drainage area, siting considerations, design procedures, and maintenance requirements. This Handbook also includes detailed guidance for infiltration testing, and basin considerations. It should be noted that project specific engineering analysis may be used to over-ride certain requirements described in this manual, as approved by the reviewing agency to ensure water quality is protected to the maximum extent practicable.

Selecting appropriate LID BMPs

LID BMPs are an effective, naturally-based form of stormwater pollutant treatment BMPs. Before selecting any particular BMPs for a site, refer to the 2018 SMR WQMP to determine LID options and requirements. The WQMP specifies particular types of LID or Treatment Control BMPs that can or must be considered for use on the project. Considerations for BMP selection include whether or not the LID BMP will maximize on-site retention of runoff, or be based on the types of pollutants that the site may generate, types of pollutants that are impairing the downstream receiving waters, and which BMPs are effective at addressing those pollutants.

Generally, infiltration BMPs have advantages over other types of BMPs, including reduction of the volume and rate of runoff, as well as full treatment of all potential pollutants potentially contained in the stormwater runoff. It is recognized however, that infiltration and retention BMPs may not be feasible on sites with high groundwater, low infiltration rates, or located on compacted engineered fill. In those situations, biofiltration based BMPs that provide opportunity for evapotranspiration and incidental infiltration may be a more feasible option. The WQMP specifies criteria that should be used to determine when particular BMPs are considered feasible.

Who should be involved in the selection, siting and design of LID BMPs?

Everyone involved with the project site development, including owners, architects, engineers, biologists, and geologists, should be informed about the proposed/required BMPs as early as possible in the planning of a project. This reduces the chance of costly redesign, the need for additional testing and produces a better and more integrated site overall. For most detention or retention basins and all infiltration BMPs, it is important that the responsible engineer/geologist be made aware of the location of BMPs, so they can make design recommendations including setbacks and perform the appropriate infiltration testing, as applicable. Landscape architects will need to know the locations and types of proposed BMPs as these might change the types of plants that can be used. Owners must be made aware of the construction costs, long-term maintenance requirements and costs, and total costs of ownership for the BMPs in order to make informed decisions during the BMP selection process.

Many of the BMP fact sheets reference the 'Engineering Authority' (EA). Who is the EA for my project?

The engineering authority for a project is the public agency responsible for reviewing and approving the proposed project. Usually the EA is the City/County wherein the project is located.

Do I need to do additional studies?

Most infiltration BMPs and basins will require a geotechnical report prepared by either a licensed geotechnical engineer, civil engineer or certified engineering geologist. The report must provide characterization of site specific soil conditions, recommendations of any required testing, and site-specific recommendations for setbacks as well as commentary on slope stability and potential offsite impacts. See Infiltration Testing Requirements and Basin Guidelines in Appendices A and C, respectively, for more information. Some sites will require other studies, such as biological resources, geomorphology, or groundwater hydrology.

Designing the BMPs

The BMPs in this Handbook are designed based on required capture volume or treatment flow rate. Volume-based BMPs are designed to capture a particular volume of stormwater runoff (referred to as V_{BMP} or the Design Capture Volume (DCV)), and either infiltrate that volume, re-use the water, or slowly and naturally filter pollutants from that stormwater, and discharge the volume within a specified "drawdown time" (the time required to regenerate the storage capacity of the BMP). Flow-based BMPs are designed to treat a required minimum flow rate of stormwater runoff (referred to as Q_{BMP}).

This Handbook contains worksheets to assist the designer in determining the required V_{BMP} and Q_{BMP} based on the location of the site. While there are likely significant direct or indirect volume reduction benefits associated with each of the included LID BMPs, these sizing worksheets are not intended to meet Hydromodification Performance Standards detailed in the SMR WQMP.

Can I make my BMP smaller?

The worksheets in this Handbook calculate the minimum required size for each LID BMP based on the amount of runoff routed to the BMP. However, early and aggressive implementation of LID Principles (site design) during the planning stages of a project can reduce the size of the effective drainage management area (DMA) for the BMP, which reduces runoff volume from the DMA, which in turn will help minimize the required size of the BMPs. To further reduce the required size, consider looking for additional ways to increase the percentage of permeable areas and porous surfaces on the site, and opportunities to drain impervious areas into pervious areas.

Can I place my BMP underground?

Underground BMPs can be an important part of an LID solution for a site. However, underground BMPs create some special challenges that must be addressed in order to provide a sustainable solution. Challenges include but are not limited to a need for effective pre-treatment, structural stability, vector control, maintenance, and the cost of replacement at the time that clogging cannot be remediated with maintenance. Special consideration needs to be given to how underground BMPs can be sustainably operated, maintained, and replaced when needed. This may include: greater emphasis on pre-treatment, inclusion of adequate inspection and maintenance features, regular inspections and effective maintenance of the BMP, and budgetary planning for major construction efforts when replacement is needed.

What are Drawdown Times?

Volume based BMPs are usually associated with a required drawdown time. The drawdown time refers to the amount of time required to regenerate the storage capacity of the BMP (i.e., drain the BMP from brim full). The specified or incorporated drawdown times are to ensure that adequate contact or detention time has occurred for treatment, while not creating vector or other nuisance issues. It is important to abide by the drawdown time requirements stated in the fact sheet for each specific BMP, or as specified by the local jurisdiction.

What is the tributary drainage area?

The tributary drainage area is the entire area that drains to the proposed onsite BMP. While small sites could be tributary to a single BMP, usually the site is broken up into several drainage management areas (DMAs), each draining to a discrete BMP. Although it is usually desirable to address offsite flows separately, if flows from offsite areas commingle with onsite flows they shall also be included in the sizing calculation. At the beginning of each fact sheet, the maximum (or minimum) tributary drainage area for each BMP is listed. The tributary areas for each BMP will be required to be clearly shown on one or more drainage exhibits. Such exhibits shall be clearly labeled to show which areas drain to which BMP.

What are pervious and impervious areas?

Project sites are made up of both pervious and impervious surfaces. The pervious portion of a site is where stormwater has the opportunity to infiltrate into the ground, such as but not limited to landscaped or natural areas. Impervious areas are where water has no opportunity to infiltrate and immediately becomes surface runoff (e.g. roofs, standard roadway pavements, concrete driveways or walkways, tightly compacted earthen materials, etc.). When a site is developed, the percentage of impervious area typically increases from the natural state. This higher impervious percentage increases the volume and flow rate of stormwater runoff.

2.0 Sizing Calculations

The following section includes sizing calculations for LID BMPs in the Santa Margarita River Watershed. There may be circumstances when flow-based Treatment Control BMPs are utilized and therefore this section also includes guidelines for calculating the design flow rate, Q_{BMP}.

2.1 Calculating V_{BMP}

Volume based BMPs and Biofiltration BMPs, including all of the BMPs in this manual, can be sized based on the design capture volume, (DCV or V_{BMP}).

The design capture volume (DCV or V_{BMP}) is based on capturing the volume of runoff generated from an 85th percentile, 24-hour storm event. Follow the steps below to calculate V_{BMP}. For convenience, these steps have also been integrated into an excel worksheet that has been provided in Appendix E of this Handbook.

1) Determine the tributary drainage area to the BMP, A_T. This includes all areas that will contribute runoff to the proposed BMP, including pervious areas, and runoff from off-site areas that commingle with on-site runoff. Calculate this area in acres.

2) Locate the project site on the full sized Isohyetal Map for the 85th Percentile 24-hour Storm Event contained in Appendix A of the SMR WQMP). These values were determined throughout Riverside County using rain gauges with the greatest periods of record. Use township, range and section information to locate the project site, and interpolate the closest value, D85, for the site. For areas near the edge of the county, extend the isohyetal lines linearly to the County boundary.

3) Determine the effective impervious fraction (I_f) for the area tributary to the BMP, using the following table:

Surface Type	Effective Impervious Fraction, I_f
Roofs	1.00
Concrete or Asphalt	1.00
Grouted or Gapless Paving Blocks	1.00
Compacted Soil (e.g. unpaved parking)	0.40
Decomposed Granite	0.40
Permeable Paving Blocks w/ Sand Filled Gap	0.25
Class 2 Base	0.30
Gravel or Class 2 Permeable Base	0.10
Pervious Concrete / Porous Asphalt	0.10
Open and Porous Pavers	0.10
Turf block	0.10
Ornamental Landscaping	0.10
Natural (A Soil)	0.03
Natural (B Soil)	0.15
Natural (C Soil)	0.30
Natural (D Soil)	0.40

If the area tributary to the BMP contains mixed post-project surface types, a composite or area-weighted average effective impervious fraction should be used. The following equation can be used for determining an area-weighted average:

$$\frac{\left[\left(I_f\right)_1 \cdot A_1\right] + \left[\left(I_f\right)_2 \cdot A_2\right] + [\ldots]}{A_T}$$

4) Calculate a runoff factor, ‘C’, using the following equation:

$$C = 0.858 \cdot {I_f}^3 - 0.78 \cdot {I_f}^2 + 0.774 * I_f + 0.04$$

5) Determine unit storage volume, V_U. This is found by multiplying the Design Storm Depth found in Step 2 by the runoff coefficient found in Step 4.

$$\mathrm{V_U} = \mathrm{D_{85}} \times \mathrm{C}$$

6) Determine V_{BMP} using the equation below or the worksheet provided in Appendix E of this Handbook. This is the volume to be used in the design of selected BMPs presented in this Handbook. Multiply the BMP tributary drainage area, A_T, by the unit storage volume, V_U, to give the BMP design storage volume.

$$\mathrm{V_{BMP}(ft^3)} = \frac{\mathrm{V_U(in - ac/ac) \times A_T(ac) \times 43{,}560(ft^2/ac)}}{\mathrm{12\ (in/ft)}}$$

If Biofiltration BMPs are used, they must be sized according to either of the following approaches:

- Have a total static storage volume including pore spaces and pre-filter detention volume, at least 0.75 times the V_{BMP} (aka DCV) not reliably retained on site. The pre-filter detention volume refers to the volume stored above the soil media and up to the maximum water quality ponding level. Pore volume refers to the volume available in the pores of the soil and/or gravel in the system.
- Treat 1.5 times the V_{BMP} (aka DCV) not reliably retained on site. This may be a volume-based or a flow-based design.

Application of these sizing methods is explained in the respective biofiltration fact sheets (3.5 and 3.6).

2.2 Calculating Q_{BMP}

While the BMPs in this Handbook are designed based on V_{BMP} (aka DCV) as discussed in 2.1 above, in some circumstances flow-based BMPs may be used. Flow-based BMPs are sized to treat the design flow rate.

Q_{BMP} is the runoff flow rate resulting from a design rainfall intensity of 0.2 inches per hour or 2 times the maximum runoff flow rate produced during 85th percentile hourly rainfall intensity. Follow the steps below to calculate Q_{BMP}.

1) Determine the tributary drainage area, A_T, that drains to the proposed BMP. This includes all areas that will contribute runoff to the proposed BMP, including pervious areas, impervious areas, and runoff from offsite areas that commingle with site runoff, whether or not they are directly or indirectly connected to the BMP. Calculate this area in units of acres.

2) Determine the effective impervious fraction (I_f) for the area tributary to the BMP, using the following table:

Surface Type	Effective Impervious Fraction, I_f
Roofs	1.00
Concrete or Asphalt	1.00
Grouted or Gapless Paving Blocks	1.00
Compacted Soil (e.g. unpaved parking)	0.40
Decomposed Granite	0.40
Permeable Paving Blocks w/ Sand Filled Gap	0.25
Class 2 Base	0.30
Gravel or Class 2 Permeable Base	0.10
Pervious Concrete / Porous Asphalt	0.10
Open and Porous Pavers	0.10
Turf block	0.10
Ornamental Landscaping	0.10
Natural (A Soil)	0.03
Natural (B Soil)	0.15

Natural (C Soil)	0.30
Natural (D Soil)	0.40

If the area tributary to the BMP contains mixed post-project surface types, a composite or area-weighted average effective impervious fraction should be used. The following equation can be used for determining an area-weighted average:

$$\frac{\left[\left(I_f\right)_1 \cdot A_1\right] + \left[\left(I_f\right)_2 \cdot A_2\right] + [\ldots]}{A_T}$$

3) Calculate a runoff factor, 'C', using the following equation:

$$C = 0.858 \cdot {I_f}^3 - 0.78 \cdot {I_f}^2 + 0.774 * I_f + 0.04$$

4) Determine the Design Rainfall Intensity using one of the following methods:
   - Use a value of 0.2 inches per hour, or,
   - Use local rainfall records, if suitable records exist, to calculate two times the maximum runoff during the 85^{th} percentile hourly rainfall intensity.

5) Determine the BMP Design Flow Rate using the equation:

$$Q_{BMP} = C \times I \times A_T$$

Where,

A_T = Tributary Area to the BMP, in acres

I = Design Rainfall Intensity, from Step 4.

C = Runoff Factor, found in Step 3

For flow-based biofiltration BMPs, the Q_{BMP} calculated above needs to be multiplied by 1.5 to meet the sizing standard for biofiltration BMPs in the SMR watershed. Flow-based biofiltration BMPs are not acceptable in all cases.

3.0 BMP Fact Sheets

This section provides fact sheets for the following thirteen types of BMPs:

3.1 - Infiltration Basins

3.2 - Infiltration Trenches

3.3 - Permeable Pavement

3.4 - Bioretention Facilities

3.5 - Biofiltration with Partial Infiltration

3.6 - Biofiltration with No Infiltration

3.7 – Regional Bioretention/Biofiltration Facility Guidance

3.8 - Bioretention Soil Media

3.9 - Tree Wells

3.10 - Extended Detention Basins

3.11 - Sand Filter Basins

3.12 - Harvest and Use

►For portability, the fact sheets for each BMP, as well as Calculation worksheets for sizing and documenting the design of these BMPs, are provided as separate downloadable files on the LID Handbook page at www.rcflood.org/NPDES/developers.aspx

BEFORE selecting any particular BMP for use on your project, review the requirements of the 2018 SMR WQMP, and the discussions in sections 1 and 2 of this Handbook. These provide important context and instructions that may dictate that particular BMPs be used.

APPENDIX A

Infiltration Testing

APPENDIX A - INFILTRATION TESTING

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 Engineering Authority (EA)[1] 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 (as directed by the EA or in the WQMP), the measured infiltration rate of the underlying soil must be determined using either the single ring infiltrometer test (as described in ASTM D 5126, Section 4.1.2.1), the double ring infiltrometer test (ASTM D 3385), 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 D 1452) 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[2]. Test pits or boring logs must be of sufficient depth to establish that a minimum of 5 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 (Sections 1.7 and 2.5). The required number of test pits or borings are 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 EA 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 1.7)

[1]*County Transportation, Coachella Valley Water District and the City Engineer for incorporated cities within the County may choose to alter these guidelines and may have different/additional requirements. These entities, along with the District, will be referred to as the Engineering Authority (EA).*

[2]*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. (Section 1.7)

5. Preliminary site grading plans shall be provided to the EA 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 ten 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 Applicability of Infiltration BMPs

The WQMP guidance applicable to a project (based on the watershed location of the project), may include specific criteria for evaluating whether infiltration BMPs are feasible for a particular project. Where the WQMP requires that infiltration testing be performed as part of an infiltration feasibility evaluation, a testing method approved by the EA shall be used. The District requires the use of the methods described in Section 2 herein. The remainder of Section 1 herein describes requirements that must be implemented whenever an infiltration BMP is to be implemented.

1.3 - 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.

1.4 - 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.

Refer also to the WQMP guidance document applicable to the project, which may identify applicability criteria for infiltration BMPs in cut conditions.

1.5 - 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.

1.6 - 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.

1.7 - 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 projects with a maximum tributary area of 5 acres 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 (RCFCD) 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.

Table 1 - Infiltration Testing Requirements							
Infiltration BMP	Testing Options	Ring Infiltrometer Tests[(1)]	Percolation Test[(2)]	Test Pits or Boring Logs[(3)]	Final Report[(4)]	Hydrology Manual[(5)]	Factor of Safety
Infiltration Trench	Option 1►	2 tests min. with at least 1 per trench	not used	1 boring or test pit per trench	Required	not used	FS = 3
	Option 2►	not used	4 tests min. with at least two per trench	1 boring or test pit per trench	Required	not used	FS = 3
	Option 3[(7)]►	not used	not used	1 boring or test pit per trench	Required	not used	FS = 6
	Option 4►	not used	not used	1 boring or test pit per site	not used	only	FS = 10
Infiltration Basin	Option 1►	2 tests min. with at least 1 per basin[(6)]	not used	1 boring or test pit per basin	Required	not used	FS = 3
	Option 2►	not used	4 tests min. with at least 2 per basin[(6)]	1 boring or test pit per trench	Required	not used	FS = 3
	Option 3[(7)]►	not used	not used	1 boring or test pit per basin	Required	not used	FS = 6
	Option 4►	not used	not used	1 boring or test pit per site	not used	only	FS = 10
Permeable Pavement	Option 1►	2 tests min. with at least 1 every 10,000 ft^2	not used	1 boring or test pit every 10,000 ft^2	Required	not used	FS = 3
	Option 2►	not used	4 tests min. with at least 2 every 10,000 ft^2	1 boring or test pit every 10,000 ft^2	Required	not used	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 1.7

(5) See Plate E-6.2 of the District's Hydrology Manual

(6) For basins 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 may be used for projects with a maximum tributary area of 5 acres only.

1.8 - 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 5 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[1] 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 factor of safety specified in Table 1 will be applied to the interpreted test results to determine the design infiltration rate for each infiltration BMP. 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 accept only the following test methods:

1. Single Ring Infiltrometer (Per ASTM D 5126), Section 2.1.1
2. Double Ring Infiltrometer (Per ASTM D 3385), 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

[1]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.

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 D 5126)

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 1.7, a final report shall be provided and, based on the test results, an infiltration rate shall be recommended.

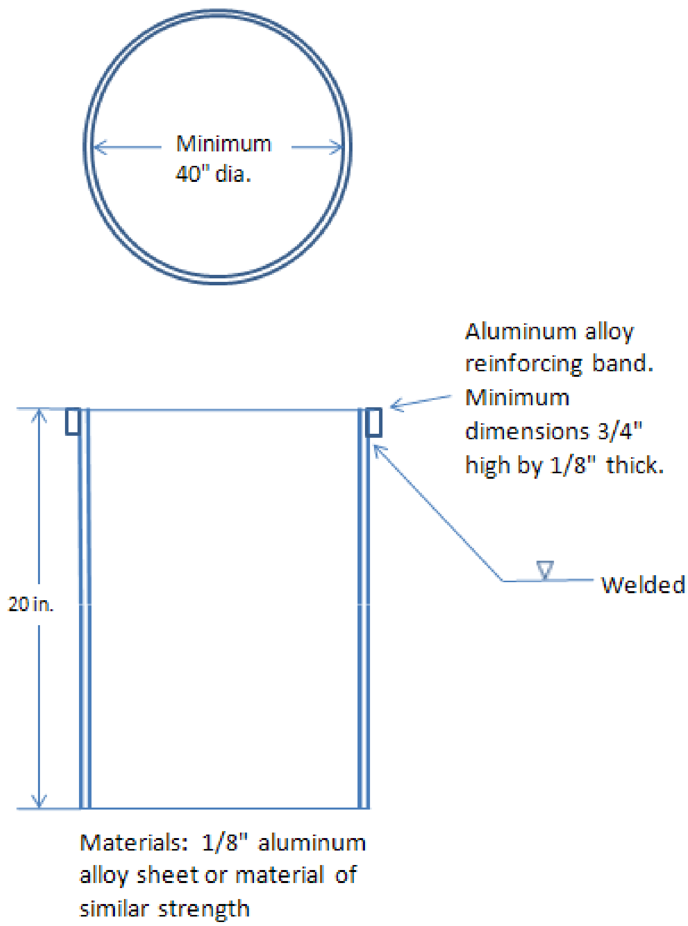


Figure 1- Single Ring Infiltrometer Construction

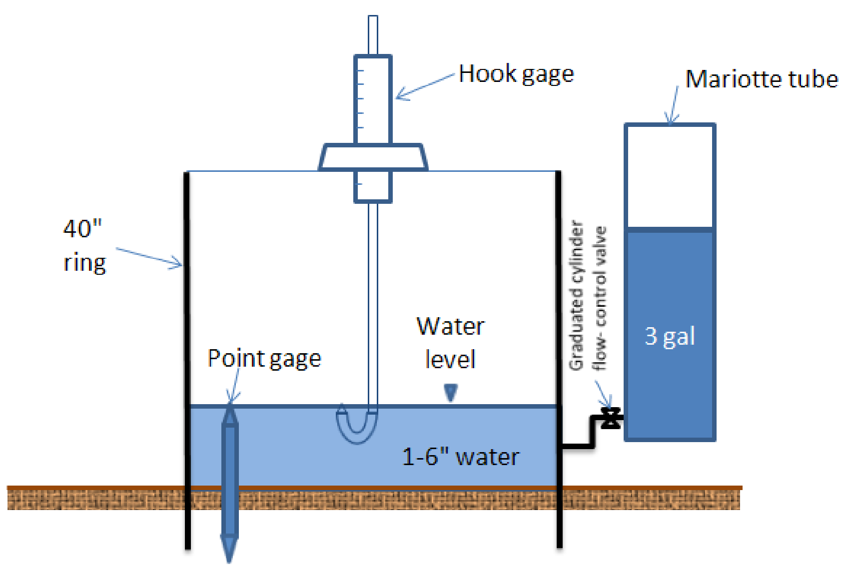


Figure 2- SIngle Ring Setup with Mariotte Tube

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	" CLOUDY, SLIGHT WIND
End	16:40	(360)	22.4	1586	64		
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

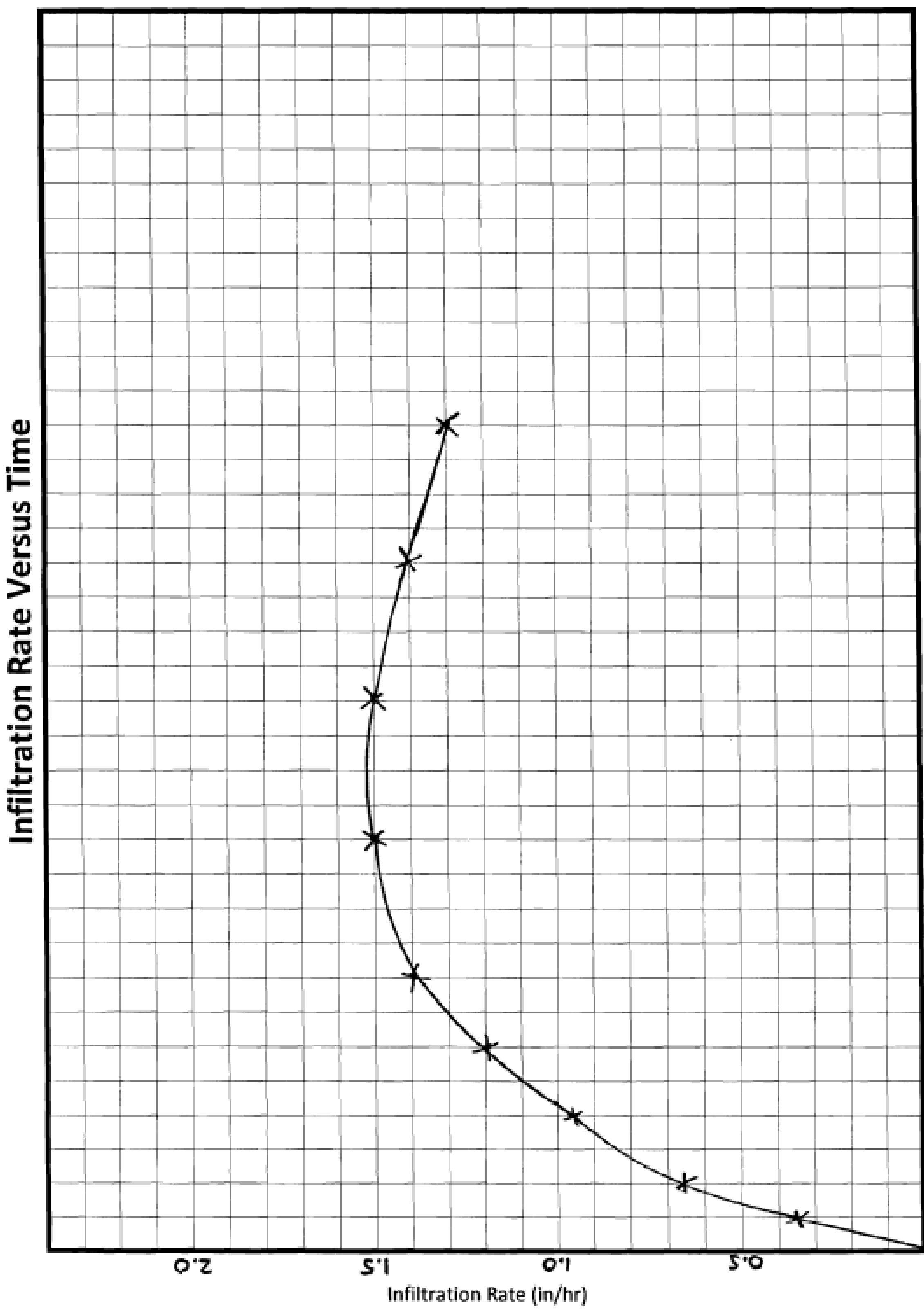


FIGURE 4 – Plot of Sample Test Data for Single Ring Infiltrometer Test

2.2.2 - Double Ring Infiltrometers

The ***double ring infiltrometer*** test (ASTM D 3385) 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 D 3385 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

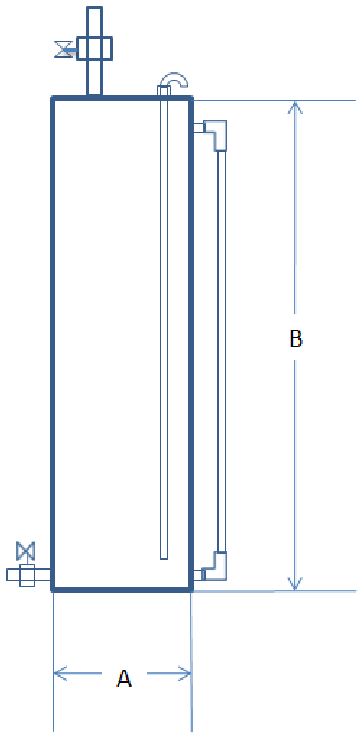
A = area of inner ring, in^2

Δt = time interval, hr.

Final Report - Ultimately, as discussed in Section 1.7, 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

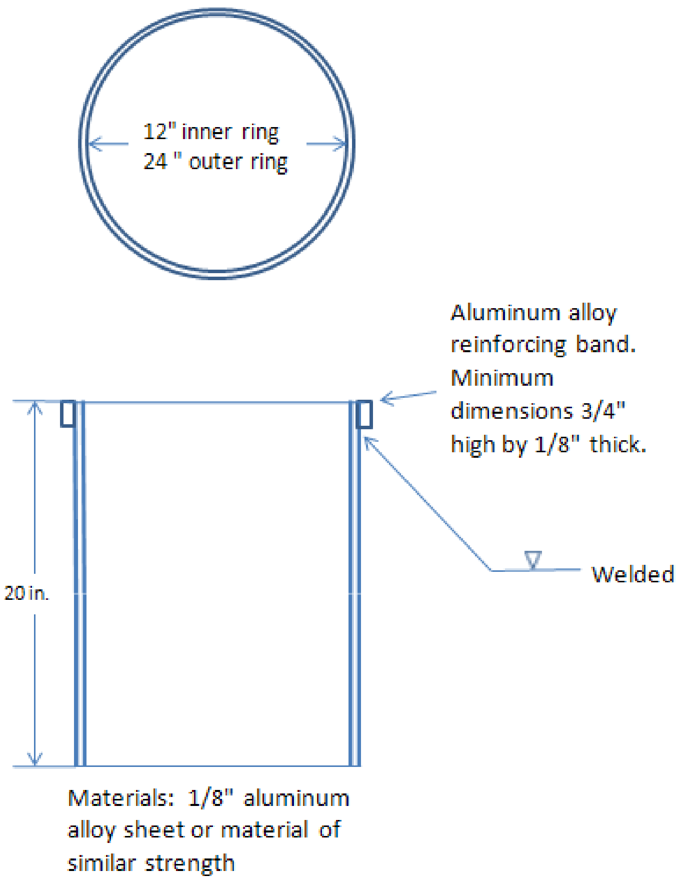


Figure 6- Double Ring Infiltrometer Construction

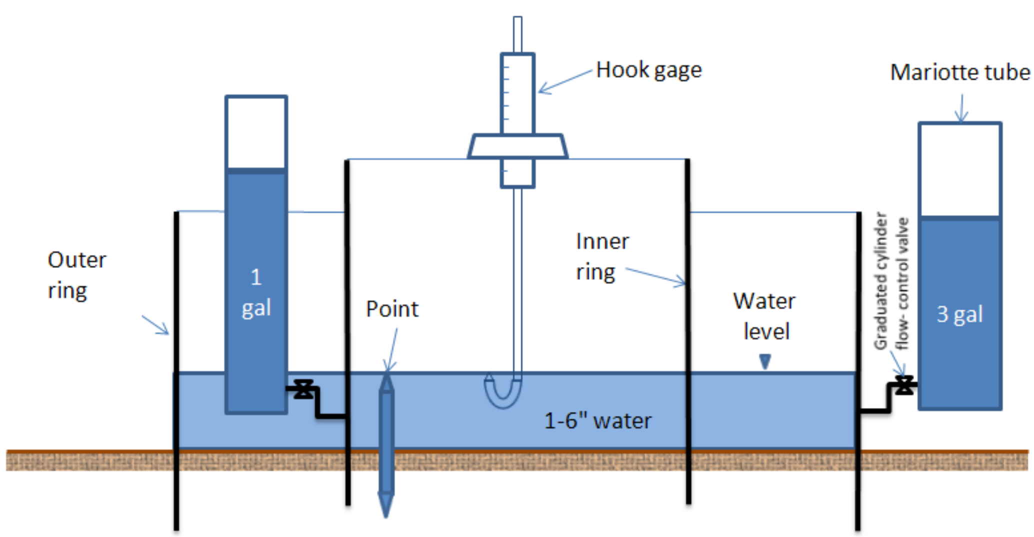


Figure 7- Double Ring Setup with Mariotte Tubes

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: Area, A_r (in^2)	Ring Data: Depth of Liquid (in)	Liquid Containers: No.	Liquid Containers: 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, $Q_f = \Delta H \times V_r$ **Infiltration Rate, $I = (Q_f/A_r)/\Delta t$**

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

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

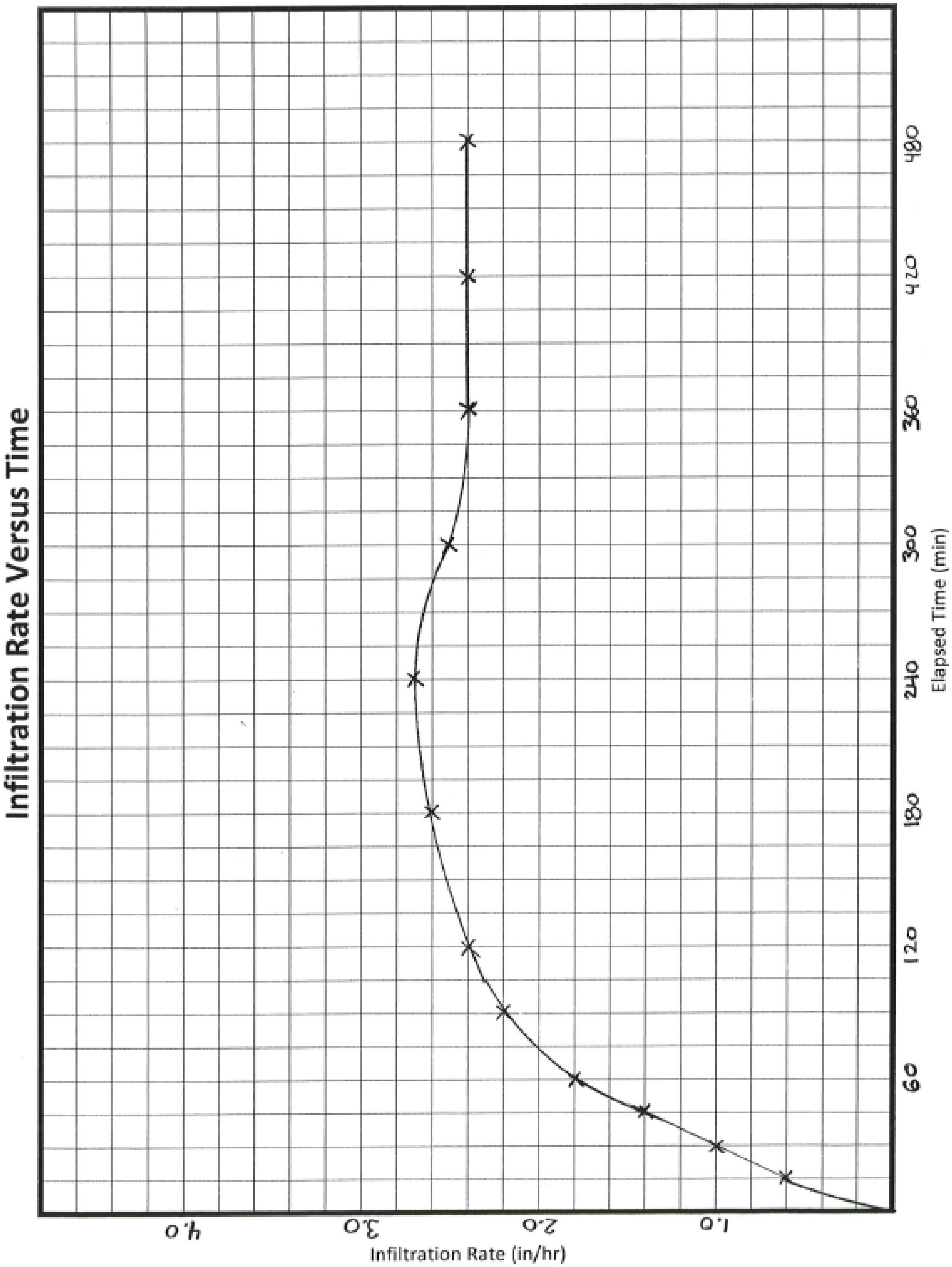


FIGURE 8 – Plot of Sample Test Data for Double Ring Infiltrometer Test

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 [1]"Porchet Method", 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 1.7, a final report shall be provided and, based on the test results, an infiltration rate shall be recommended.

[1]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 Chapter.

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 1.7.

Shallow Percolation Test (less than 10 feet)

Test Preparation

1.) The test hole opening shall be between 8 and 12 inches in diameter or between 7 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 5 times the hole's radius.

3.) The bottom of the test hole shall be covered with 2 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 5 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 5 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 5 times the hole's radius (H/r>5) above the gravel at the bottom of the test hole prior to each test interval.

- In **<u>sandy soils</u>**, when 2 consecutive measurements show that 6 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 twelve 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 6 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.

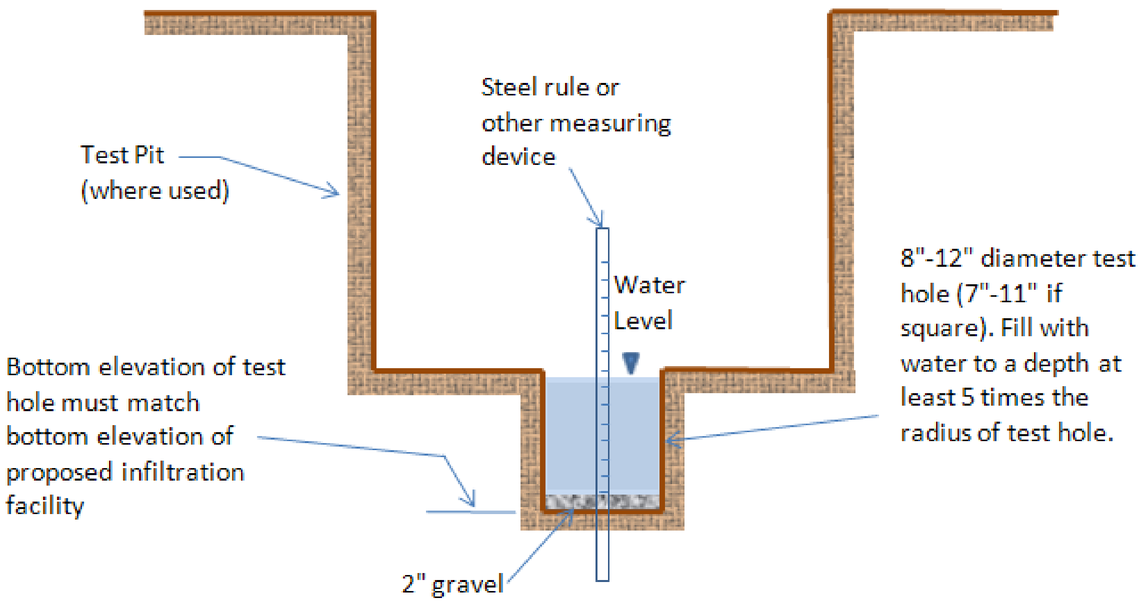


Figure 9- Test Pit for Shallow Percolation Test

Deep Percolation Test (Depths 10-40 feet)

Test Preparation

1.) Borehole diameter shall be either 6 inch or 8 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 5 times the hole's radius.

3.) The bottom of the test hole shall be covered with 2 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 5 gallon bottle of clear water supported over the hole so that the water flow into the hole holds constant at a maximum depth of 4 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 5 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 2 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.

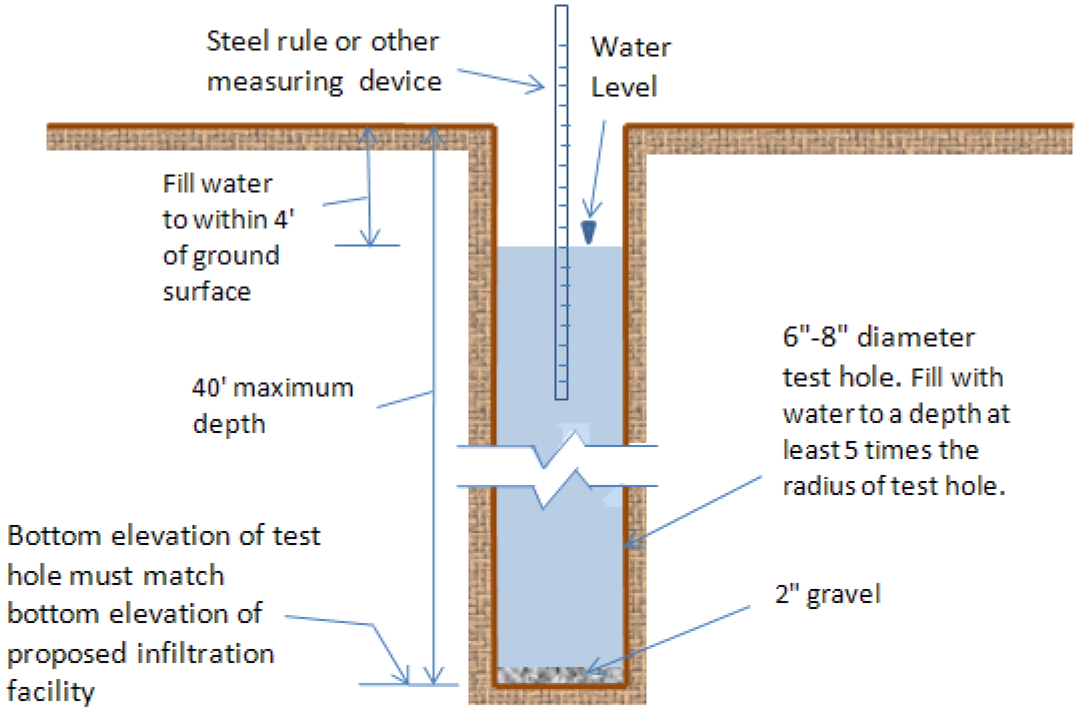


Figure 10- Test Pit for Deep Percolation Test

Test Procedure

Carefully fill the hole with clear water to a maximum depth of 4 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 5 times the hole's radius (H/r>5).

- In **<u>sandy soils</u>**, when 2 consecutive measurements show that 6 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 **<u>non-sandy soils</u>**, the percolation rate measurement shall be made on the day following initiation of the pre-soak as described in Item #5 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 5.0 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. [1]**The data collected at the final interval is as follows:**

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

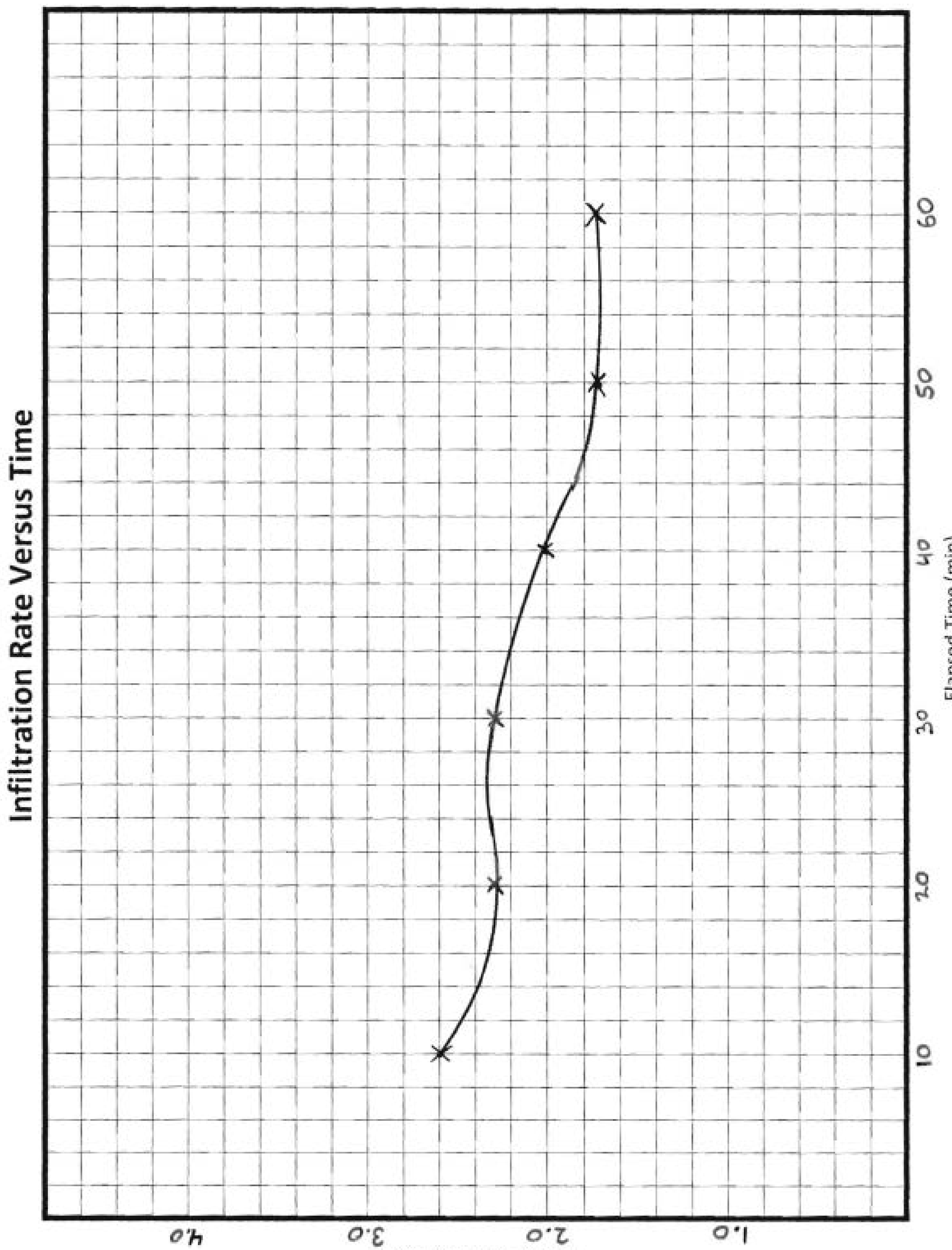


FIGURE 11 – PLOT OF CONVERTED PERCOLATION TEST DATA

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. [1]USBR 7300-89 is an in-hole hydraulic conductivity test performed by drilling test wells with a 6-8 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:

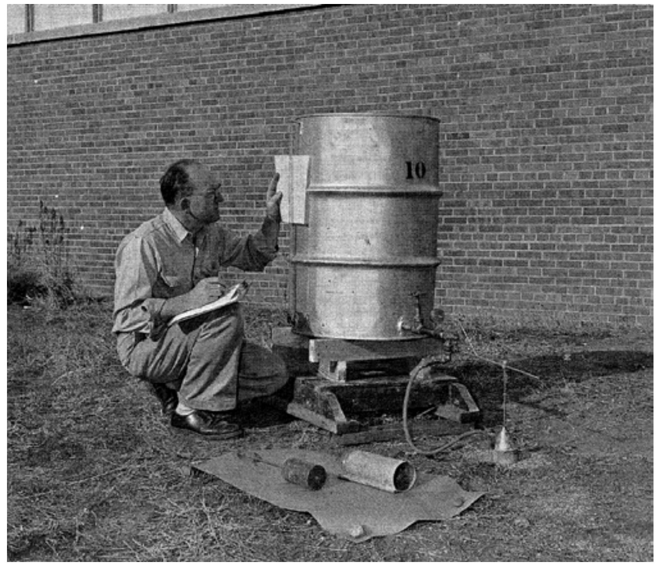


Photo 7 - Typical Well Permeamater Test Installation

Condition I: Typical condition (See Figure 12). The distance between the historical high water mark[2] 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 5 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

[1]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.

[2]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.

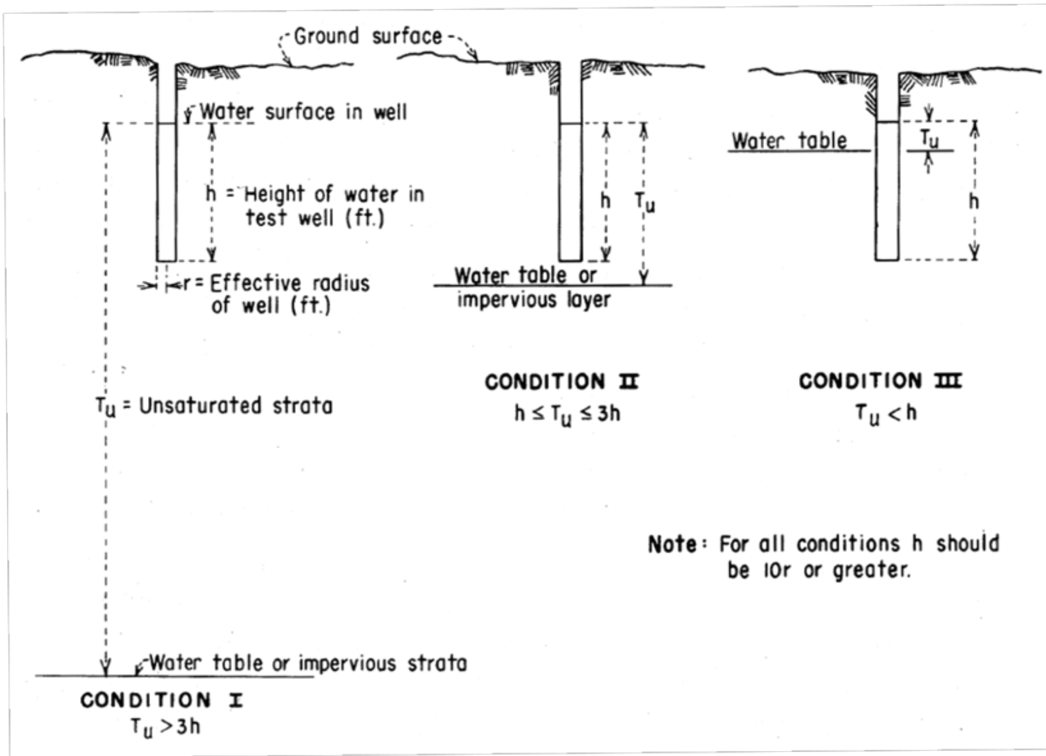


Figure 12 – Site Conditions Govern Procedure to be Used

Condition II: The distance between the historical high groundwater mark[1] 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[1] 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 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 or 5 feet from impervious strata.

Final Report - Ultimately, as discussed in Section 1.7, a final report shall be provided and, based on the test results, an infiltration rate shall be recommended.

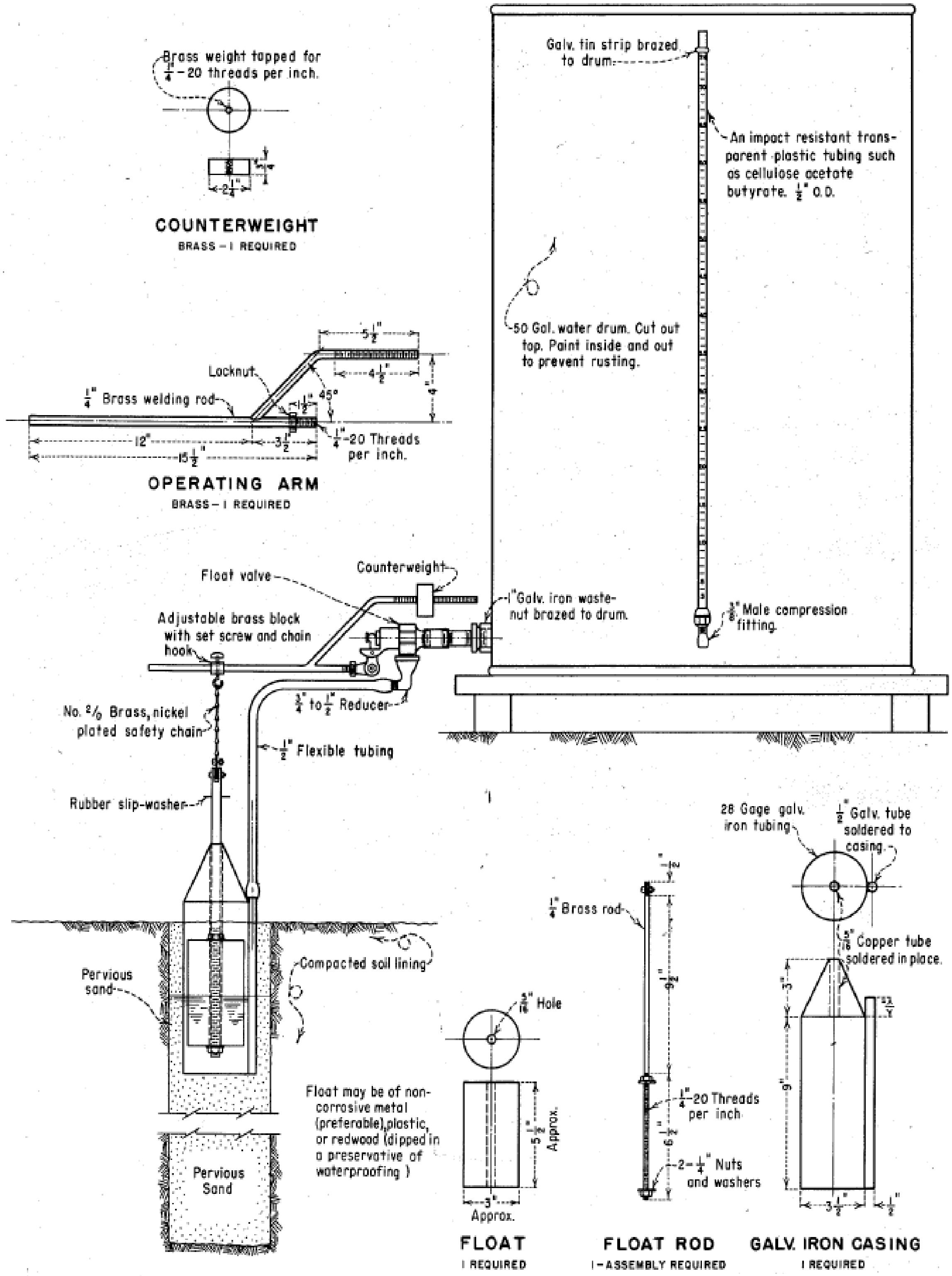


Figure 11 – Well Permeameter Test Equipment

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 D 1452) 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 5 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[1].

Photo 8- Auger Boring Rig

Photo 9 – Test Pit Excavation

[1]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.

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 D 5126, Section 4.1.2.1), the double ring infiltrometer test (ASTM D 3385), 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 D 1452) 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 5 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[1] (Section 1.7 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 District or other EA 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 1.7)

___ 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 1.7)

___ Preliminary site grading plans shall be provided to the District or other EA 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)

[1]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.

APPENDIX B

Underdrains

APPENDIX B – UNDERDRAINS

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

Underdrain Material Types

Underdrain pipe shall be 6-inch 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 BMP's 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. or greater)	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. Clean-outs, with diameters equal to the underdrain, should extend 6 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 Hydraulic Conditions of Concern (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.

APPENDIX C

Basin Guidelines

APPENDIX C – BASIN GUIDELINES

This appendix is broken up into two sections. Section 1 presents guidelines and standards for the design and maintenance of water quality and increased runoff basins used within Riverside County. Applicable water quality basins include infiltration, sand filter and extended detention basins but do not include Bioretention BMPs. Section 2 is devoted to guidelines and standards for debris basins. Regional Basins are only loosely governed by this document and are largely considered on a case-by-case basis.

These guidelines are *intended* to be used on both private and public facilities throughout Riverside County and *shall* be adhered to for all facilities to be maintained by the Riverside County Flood Control and Water Conservation District (District). It is anticipated that County Transportation, Coachella Valley Water District and the City Engineer for incorporated cities within the County may choose to alter these guidelines and may have different/additional requirements. These entities, along with the District, will be referred to as the Engineering Authority (EA). Similarly, County or City Planning Departments, Parks Departments and Parks Districts may also have different/additional requirements. These entities will be referred to as the Planning Authority (PA). Both the EA and PA should be consulted regarding their specific requirements.

Section 1- Detention and Water Quality Basins

1.1 General Criteria

Off-line versus In-stream Mitigation – All water quality mitigation basins must be flow-through. In-stream mitigation is extremely difficult to accomplish unless the basin is designed to accommodate all upstream tributary area and to mitigate for all impacts due to upstream development. Therefore most EAs will not allow in-stream water quality mitigation basins. It shall be noted that while flow mitigation BMPs may be allowed to be constructed within "jurisdictional waters", water quality mitigation BMPs will not be permitted.

Dam Safety Compliance – Basin designs that would be considered "jurisdictional" and fall under the Division of Safety of Dams (DSoD) review are not recommended.

Standard Details - Most EAs would prefer standardization of elements of outlet structures that are likely to wear (e.g., trash racks). Outflow control structures shall be designed in accordance with the EA's standards unless site-specific conditions preclude it. The District requires the use of Standard Drawing WQ501 for most basins. However a modified District CB110 overflow outlet is recommended for infiltration and sand filter basins. Minor modifications to provide supplemental hydraulic routing characteristics above the water quality storage volume are acceptable.

General Sizing Criteria – These guidelines relate to the basic features to be included in the various types of basins and the general geometrics of the basins design criteria. This appendix does not include the volumetric sizing of facilities. Follow the appropriate increased runoff guidelines or BMP fact sheet sizing.

Geotechnical Reports – A geotechnical report prepared by either a licensed geotechnical engineer, civil engineer or certified engineering geologist is required for all basins. The minimum content of the Geotechnical report shall include the following:

- Slope stability - Discussion shall include the affect the basin may have on the stability of adjacent slopes as a result of the basin's proposed location.

- Compaction, cut and fill - Issues due to soil compaction and/or cut and fill conditions with regards to the safety and effectives of the basin shall be discussed.

- Setbacks from buildings, slopes, wells - The report shall include recommendations for the minimum setback required from buildings, onsite walls, and slopes. In addition, the report shall determine the location of any pre-existing wells (onsite or offsite) and clarify that the minimum 100 foot horizontal setback has been maintained.

- Embankment design - For embankments over 5 feet in height, the geotechnical report shall include recommendations as to its construction and clarify that the embankment meets the requirements herein.

- Boring logs - Boring logs shall be provided within the report and a discussion of their findings included. Any subsurface conditions which may be pertinent to the safety and effectiveness of the basin shall be discussed.

- High Groundwater Level - The historic high groundwater level shall be determined. It shall be clarified that a minimum 10 foot vertical separation from the bottom of the basin to the top of the historic high groundwater level will be maintained.

In addition, where infiltration basins are to be utilized on the site the following topics shall be discussed:

- Existing Conditions (i.e., legacy pollutants) - Where existing soil contains unusually high levels of pollutants, the report shall clarify the extent of the pollution, mitigation efforts and the viability of using an infiltration BMP effectively as a result.

- Infiltration Testing - The use of infiltration BMPs may require an infiltration rate be established as described in Appendix A, "Infiltration Testing Guidelines". An infiltration testing report may be required and documented as described therein

Parking Lot Detention - Parking lots <u>shall not</u> be used to provide additional surface (above ground) storage for either water quality BMPs or to address HCOCs.

Seeps and Springs- Intermittent seeps along cut slopes are typically fed by a shallow groundwater source (interflow) flowing along a relatively impermeable soil stratum. These flows are precipitation driven and should discontinue after a few weeks of dry weather. No special provisions are needed when directing these flows through the basin. However, more continuous seeps and springs, which extend through longer dry periods, are likely from a deeper groundwater source. When continuous flows are intercepted and directed through basins, adjustments to the approved facility design may be required to account for the additional base flow (unless already considered in design).

Privately Owned Basins - All of the criteria herein apply to privately maintained basins except that retaining walls may be used for a portion of interior slopes. Privately owned basins are only acceptable for commercial projects, multi-family residential projects and single family residential communities with a viable maintenance mechanism. Retaining walls may not be used to support water impounding embankments. Retaining walls shall not exceed one third of the outside perimeter of the basin. Detailed structural design calculations must be submitted with every retaining wall proposal. A fence shall be provided along the top of the wall. **The use of retaining walls in a basin requires approval prior to tentative project approval.** The EA or PA may reject the proposed use of retaining walls due to aesthetic and maintenance concerns relating to nuisance and graffiti abatement.

1.2 - Basin Grading Parameters

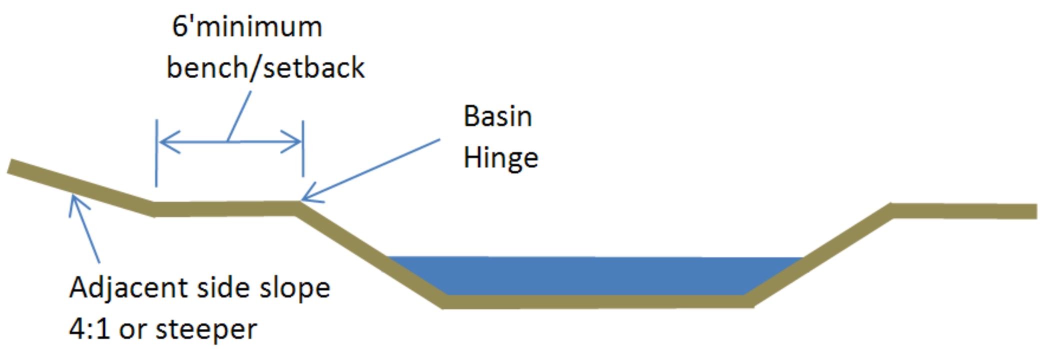
Basins must meet the following requirements for side slopes, fencing, and embankments:

Interior Side Slopes - At least 50 percent of the facility perimeter shall have interior sides no steeper than 4H:1V and in no case steeper than 2H:1V (even if fenced) to minimize safety risks. Side slopes shall be no steeper than 4H:1V whenever adjacent to down-gradient external property lines, roadways, sidewalks and trails.

Embankments - Embankment fill slopes (external and internal) may be no steeper than 4:1 with no exceptions. Basin embankment height will be based on the vertical distance from the design overflow water surface (typically the spillway invert elevation) to the lowest downstream toe of embankment fill. Basin embankments higher than 5 feet shall require design by a geotechnical engineer and shall have a top width not less than 20 feet. For embankments 5 feet or less in height, the minimum top width shall be 6 feet. Embankments for water quality basins may not exceed 3 feet in height.

Setbacks - All basin grading impacts shall be set back a minimum of 6 feet from down-gradient external property lines. This requirement applies to both the top of a cut-slope and the toe of any exterior slope embankment, along with rip-rap energy dissipaters relative to the property line (excluding road right of way). The cut-slope setback requirement is intended to avoid situations where future offsite grading/cut-slopes could turn an incised basin into an embankment-impounded reservoir. For all cases, depending on the amount of discharge and site characteristics, additional setback may be required unless appropriate easements are secured from the affected property owner(s).

There shall be a minimum 6 foot setback between a basin and an adjacent slope 4:1 or steeper measured horizontally from the basin hinge to the toe of the slope.



Forebay - A forebay shall be placed at each inlet to the basin to allow for the settlement and collection of larger particles. A relatively smooth concrete bottom surface should be provided to facilitate mechanical removal of accumulated sediment, trash and debris. A rock or concrete berm separates the forebay from the remainder of the basin. The forebay's design volume must be from 3 to 5% of the design volume, with the exception of infiltration and sand filter basins whose forebays should be 0.5% of the design volume. A full height notch-type weir shall be made through the berm to convey water to the main body of the basin. This notch shall be offset from the inflow streamline to prevent low-flows from short circuiting.

Basin Floor Slopes - Surface slopes should be kept at a minimum to allow for as much infiltration/groundwater recharge as is possible while still meeting vector concerns. All detention and extended detention basins shall have transverse and longitudinal bottom surface slopes of 1% minimum. For infiltration and sand filter basins, the basin floor should be level.

Gravel filled low-flow trench

Dry Weather Flow Management – All increased runoff or extended detention basins (**excluding** infiltration or sand filter basins) shall be designed to accommodate dry weather flows without impairing wet weather function or creating potential nuisance or maintenance issues. The basin shall have a network of gravel filled low-flow and collector trenches covering the entire basin floor area along with a sand filter drain adjacent to the outlet structure. See Figure 1 on following page.

A 48-inch wide by 24-inch deep low-flow trench conveys flow from the forebay to the filter drain. With a mild longitudinal slope of at least 1% to promote infiltration, the unlined low-flow trench shall be filled with 2" gravel (ASTM No. 2 or similar) to the finished surface and shall not use perforated subdrains.

Collector trenches beneath the top stage shall be arranged in accordance with Figure 1 with a maximum slope of 0.5% to promote infiltration and must extend from the low-flow channel to the toe of the basin side slopes. They shall be 18-inches wide by 24-inches deep and filled with 2" gravel (ASTM No. 2 or similar) to the finished surface. The collector trenches shall not have perforated subdrains and shall be constructed with a maximum spacing of 25 feet on center. See Figure 1 on following page.

A sand filter drain shall be constructed at the low point (or bottom-stage) of the basin adjacent to the outlet structure. To avoid clogging at the lowest orifice of the outlet structure, the top of the filter drain is offset below the lowest orifice of the outlet structure by 0.33 feet (4 inches). The sand filter drain shall include an 18 inch layer of sand (fine aggregate per

ASTM C-33) over a 10-inch gravel subdrain system and shall line the entire bottom stage. The total depth of the sand filter drain, D_{FD}, shall therefore not be less than 2.33 feet. See Appendix B for standard subdrain construction. The filter drain's design volume must be a minimum of 0.5% of V_{BMP} and the minimum bottom stage area is $A_{BS} = V_{BMP} / D_{BS}$.

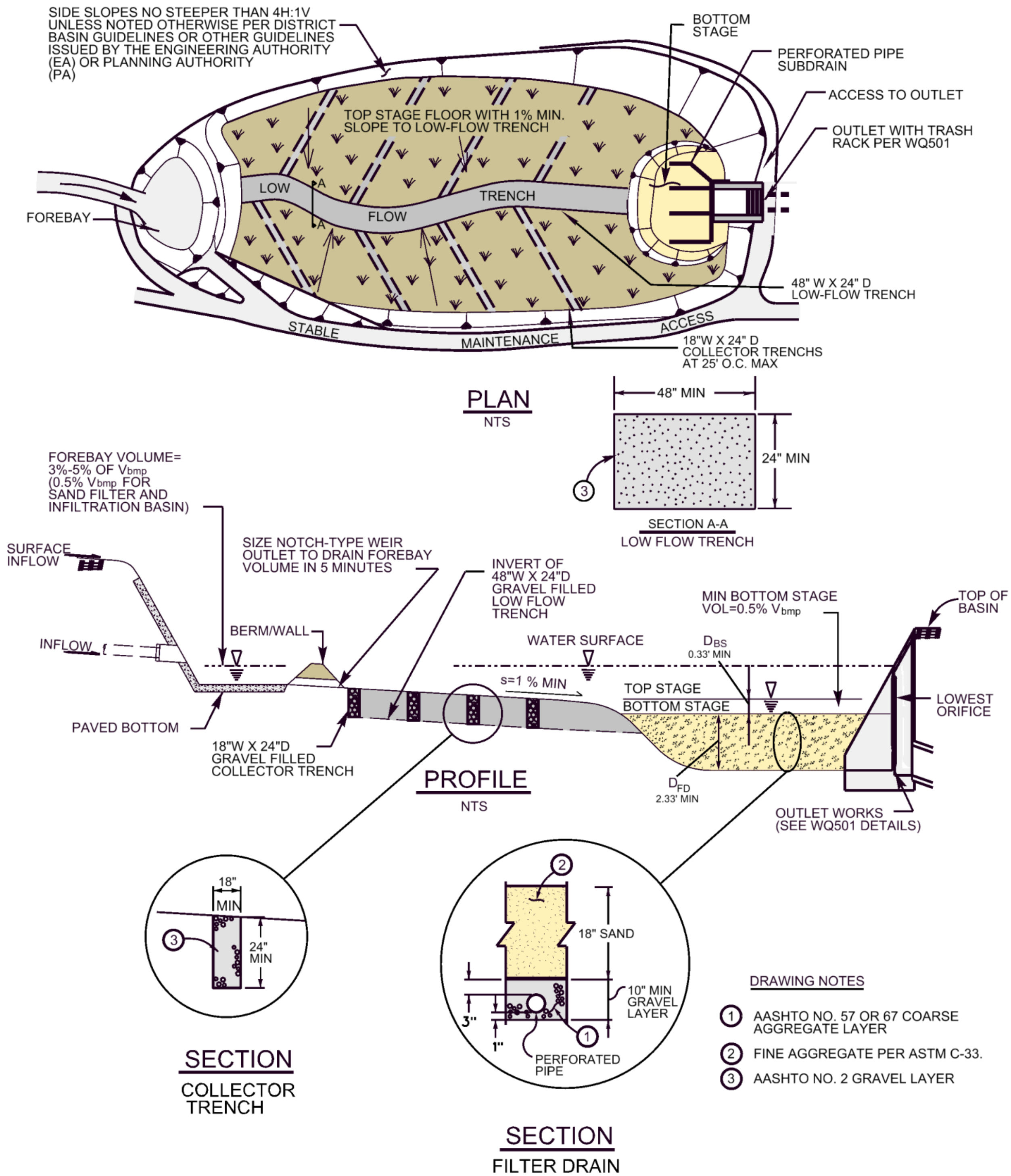
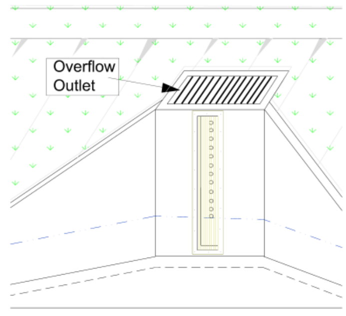


Figure 1 –Dry Weather Management Features

Outlet Structure and Spillway - Outlet structures shall conform to District Standard Drawing WQ501 unless approved in advance by the EA. This standardization is important in order to provide for efficient maintenance.

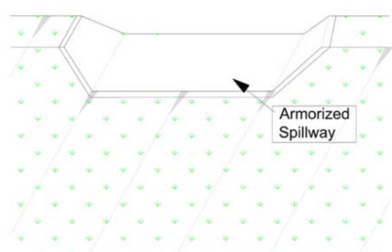
a. **Water Quality Outlet Trash Rack/Screen** - The outlet's orifice plate shall be protected with a conforming trash rack with at least six square feet of open surface area or 25 times the total orifice area, whichever is greater. The rack shall be adequately secured to prevent it from being removed or opened when maintenance is not occurring.

b. **Overflow Outlet** - In all basins, a primary overflow (usually integrated into the control structure) must be provided to pass flows greater than the design volume up to the 100-year event. The design must provide controlled discharge directly into the downstream conveyance system or an acceptable discharge point.



c. **Emergency Overflow** - In addition to the above overflow requirements, basins must have an emergency overflow escape path sized to safely pass the 100-year tributary developed peak flow in the event of total control structure failure (e.g., blockage of the control structure outlet pipe) or extreme inflows. Emergency overflow pathways are intended to control the location of basin overtopping and direct overflows back into the downstream conveyance system or other acceptable discharge point.

d. **Emergency Overflow Spillway -** Basins with constructed embankment over 3 feet in height and for BMP embankments of any height, or located on grades in excess of 5% must provide an emergency overflow spillway structure. The emergency overflow spillway must be designed to pass the 100-year developed peak flow, with a minimum 12 inches of freeboard, directly to the downstream conveyance system or an acceptable discharge point. The emergency overflow spillway shall be armored full width, beginning at a point midway across the berm embankment and extending downstream to an adequate outlet point. Design of emergency overflow spillways generally requires the analysis of a broad-crested trapezoidal weir.



Access Roads and Ramps - Maintenance access road(s) shall be provided to the top of the control structure and other drainage structures associated with the basin (e.g., inlet/forebay, emergency overflow or bypass structures). All basins shall have unobstructed access from a public street (see Section 1.4, "Right-of-Way") with commercial size curb cut-outs and driveway approaches. Flood control basins designed to attenuate the 100 year flood event shall have an access road around the entire basin. Manhole and catch basin lids should be within or at the edge of the access road and shall be at least three feet from a property line. Rims shall be set at the access road grade.

On large, deep basins (at least 1500 square feet bottom area, measured without the ramp, and over 4 feet deep), an access ramp must extend to the basin bottom at the forebay for removal of sediment with a trackhoe and truck. This is necessary so truck loading can be done in the basin bottom.

However, on small deep basins (less than 1500 square feet, but over 4 feet deep), the truck can remain on the ramp for loading. As such, the ramp may end at an elevation up to 4 feet above the basin bottom provided the basin side slopes are 4:1 or flatter.

On small shallow basins (less than 1500 square feet bottom area, and 4 feet deep or less), a ramp to the bottom is not required if the trackhoe can load a truck parked at the basin edge (trackhoes can negotiate mild interior basin side slopes).

No ramp is required for any basin 4-feet or less in depth if vehicular access is provided to the top of slope at the forebay and the side slopes are 4:1 or flatter. (Depth trigger for ramp is measured from top of slope adjacent to forebay invert.)

Design of access roads and ramps shall meet the following design criteria:

a. Maximum grade (measured along ramp centerline) shall be 15% for asphalt or concrete paving and 10% for soft surface or modular grid paving.
b. Inside turning radius shall be 35 feet, minimum.
c. Fence gates shall be located only on straight sections of road.
d. Access roads shall be constructed with an asphalt, concrete, 3-inch layer of compacted Class 2 aggregate road base material, decomposed granite or modular grid pavement.
e. Access roads and ramps shall be 15 feet in width on curves, 12 feet on straight sections. A paved apron shall be provided where access roads connect to paved public roadways.

1.3 - Landscaping

Landscaping will likely be required by the Planning Authority. Landscaping requirements shall be in accordance with Riverside County Ordinance 859 or equivalent agency ordinance. Care must be taken to ensure that landscaping does not hinder maintenance operations.

a. Facilities shall be designed so that they do not require mowing. Where mowing cannot be avoided, facilities shall be designed to require mowing no more than once or twice annually. A 6-foot minimum width must be provided to allow a mower to pass (see Figure 2).

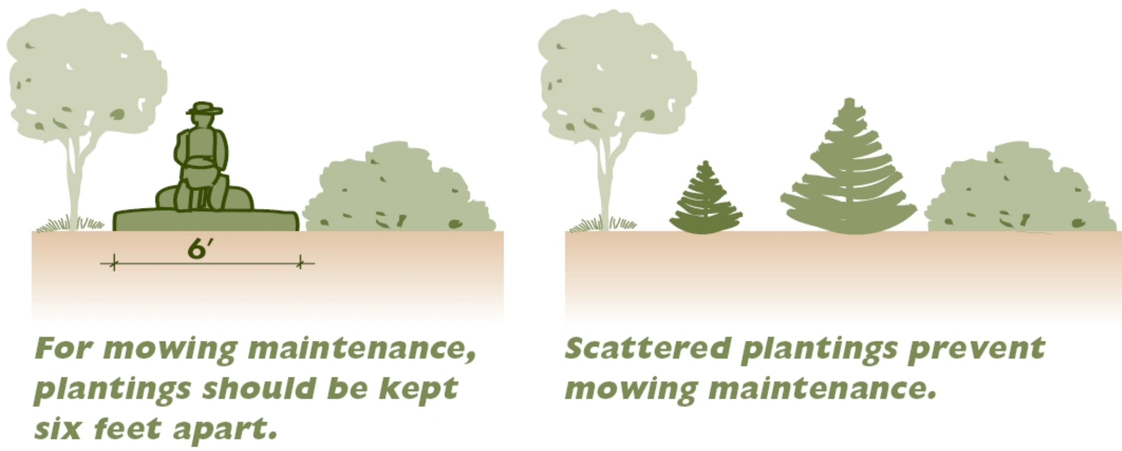


Figure 2- Landscaping setbacks (Source: King County WLR)

b. Turf and lawn areas are not allowed for publicly maintained basins unless an appropriate landscape maintenance entity is identified.

c. Planting is restricted on embankments that impound water either permanently or temporarily during storms (see figure 3). This reduces the likelihood of blown down trees, or the possibility of channeling or piping of water through the root system, which may contribute to dam failure on embankments that retain water.

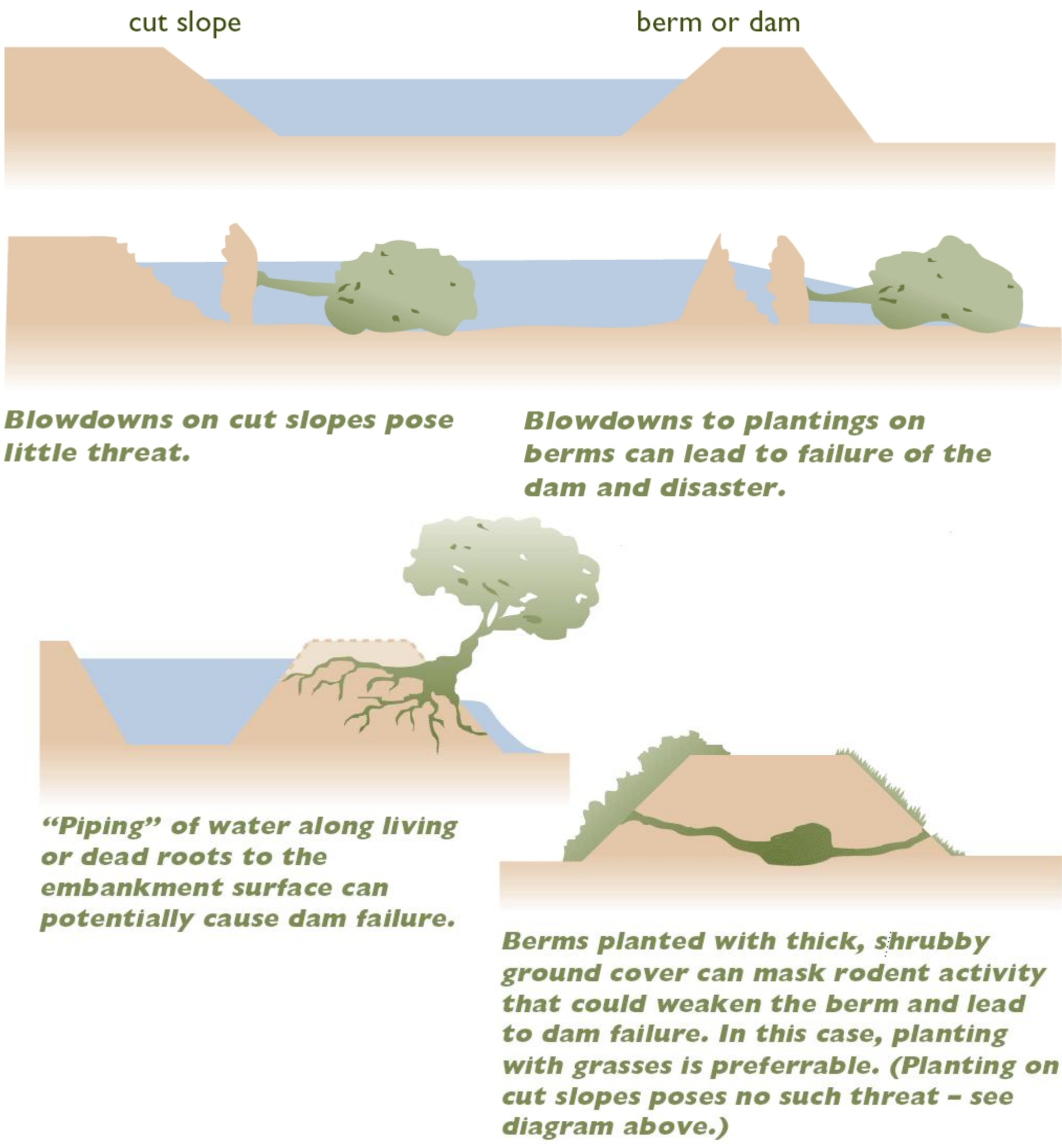


Figure 3 - Hazardous Landscaping Practices (Source: King County WLR)

Note: *This restriction does not apply to cut slopes that form basin banks, only to embankments.*

d. No trees or shrubs may be planted within 10 feet of inlet or outlet pipes or from manmade drainage structures such as spillways or flow spreaders.

e. Trees with roots that seek water, such as willow or poplar, should be avoided within 50 feet of pipes or manmade structures.

f. Evergreen trees and others that produce relatively little leaf-fall (such as locust) are preferred in areas draining to the basin. Trees should be set back so branches do not extend over the outlet structure area of the basin (to help prevent clogging). Drought tolerant species are recommended.

g. Trees or shrubs may not be planted on portions of water-impounding embankments taller than four feet high. Only grasses may be planted on embankments taller than four feet.

1.4 - Additional Requirements

Fencing Criteria - The requirements for slopes and fencing are intended to discourage access to portions of a basin where steep side slopes (steeper than 4:1) increase the potential for slipping into the basin, and to allow easy egress for those who have fallen with slopes that are mild enough (flatter than 4:1 and unfenced) to allow for easy escape. If the basin will hold water deeper than 2 feet, a physical barrier as demarcation of the basin limits is required:

a. Where interior slopes are steeper than 4:1, the barrier shall be a fence 6 feet in height (see District Standard Drawing M-801 for chain link fence details). In joint use ventures where a special district or agency has agreed to maintain landscape facilities, tubular steel fencing such as that meeting Valley Wide Recreation and Parks District landscape standard LC-10 is also acceptable. Functionally equivalent designs may be acceptable on a case by case basis.

b. Where interior slopes are 4:1 or milder, the physical demarcation shall be (3-foot minimum height) vinyl or PVC rail fence, post-and-cable, masonry wall, or densely planted hedges. Functionally equivalent designs may be acceptable on a case by case basis.

c. If the side slopes undulate, and segments of the slope are steeper than 4:1, the barrier standard from “b.” above may be used in place of the 6-foot fence for the short lengths of slope as specified here: The barriers described in “b.” may be used for sections of 2:1 slope not to exceed 20 lineal feet and sections of 3:1 slope not exceeding 50 lineal feet.

d. If required, fencing shall be placed at or above the overflow water surface. Side slope and attendant fencing requirements are not applicable to slopes above the overflow water surface.

Gates - Vehicular access shall be limited by a double-posted gate if a fence is required, or by bollards. Access road gates shall be 14 feet in width consisting of two swinging sections 7 feet in width (see the District’s Standard Drawing M-801 for details). Alternately, two fixed bollards on each side of the access road and two removable bollards equally located between the fixed bollards may be used. Additional vehicular access gates may be required as needed to facilitate maintenance access. Pedestrian access gates (if needed) shall be 4 feet in width.

Signage - All basins to be maintained by the District shall have a sign placed for maximum visibility from adjacent streets, sidewalks, and paths. The sign shall meet the design and installation requirements illustrated in Figure 4.

Right-of-Way - Basins shall not be located in a dedicated public road right-of-way. Publicly maintained basins shall be in a lot dedicated to the public. Any lot not abutting the public right-

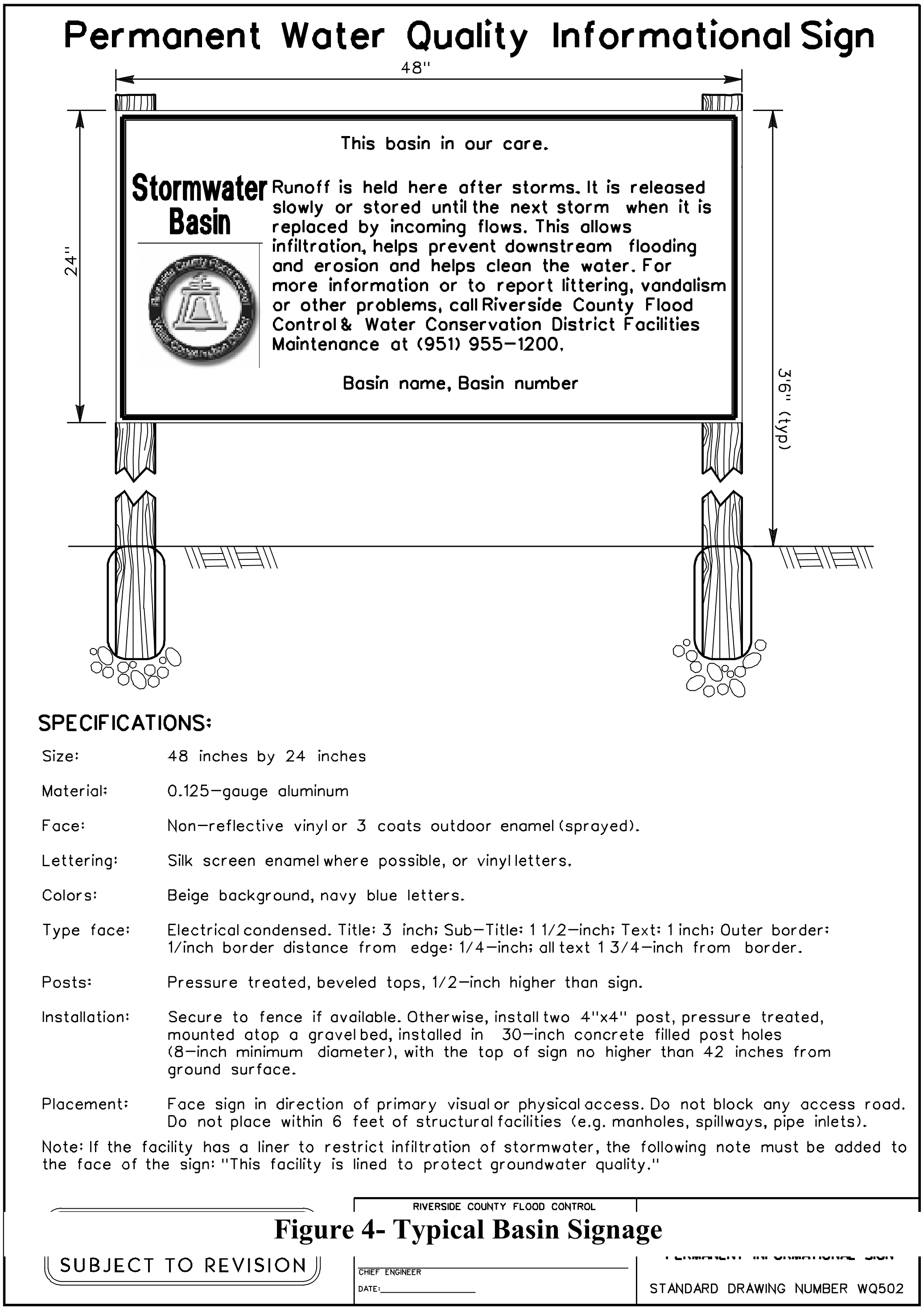
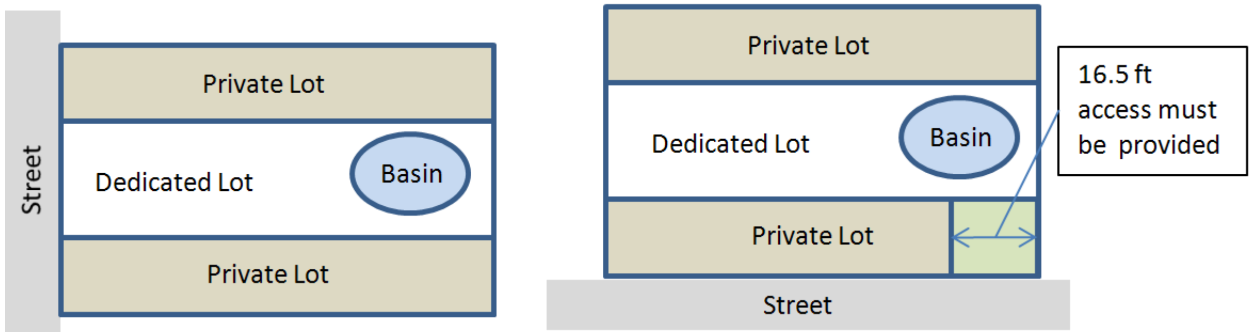


Figure 4- Typical Basin Signage

of-way will require a 16.5-foot wide extension of the lot to an acceptable access location.



Case 1 - Access available to public road right-of-way.

Case 2 - Access must be acquired through adjacent lot to public road right-of-way.

1.5 - Basins in Recreational Spaces

Any basin site with a bottom surface area larger than one acre will likely be required to incorporate active use area and shall be designed only after consultation with the PA to establish site-specific guidance which may increase the total facility footprint.

If multiple uses are being contemplated, consider the following:

- Place the active use areas such as ballparks, playing fields, and picnic areas above the water quality design volume (V_{BMP}) ponding limit.
- Use a multiple-stage detention basin to limit inundation of passive recreational areas to one or two occurrences a year.
- Side slopes shall not exceed 25% (4:1) unless they are existing, natural, and covered with vegetation.
- Locate the basin in a separate lot.
- Incorporate a bypass system or emergency overflow pathway that does not present a safety hazard or discharge into active recreation areas.
- The basin shall be landscaped in a manner to enhance passive recreational opportunities such as trails and aesthetic viewing. Inquire with the PA whether the basin can be compatible with the open space value and functions.

If the criteria above are met, projects may be able to receive some reduction in required onsite recreational space if approved in advance of tentative project approval by the PA.

Section 2 - Debris Basins

Debris basins differ from stormwater detention and water quality basins in that they are not intended to detain flows or to mitigate pollutants (other than debris). They are simply utilized to collect large debris from storm flows for later removal. The guidelines in this section apply to debris basins only.

Site access – Debris basins shall have unobstructed access from a public street (see Section 1.4, "Right-of-Way") with commercial size curb cut-outs and driveway approaches.

Fencing – The entire facility shall be enclosed with 6-foot high chain link fencing and 14-foot high double drive gates. Where the perimeter fencing crosses a streambed, cable or barbed wire fencing across streambed will be provided.

Maintenance access - Maintenance access shall extend around the entire perimeter of the facility. Roads shall be a minimum of 15 feet wide (20 feet wide if on an embankment of 3 feet or higher). The minimum design turning radius shall be 35 feet. Ramps shall be a minimum of 15 feet wide with a maximum longitudinal slope of 10%. Both roads and ramps shall be surfaced (full width) with 3" of compacted Class 2 base material.

Basin Cut/Fill slopes – All basin slopes shall not be steeper than 3:1.

Stockpile/Staging Area – Shall be situated immediately adjacent to the basin. The minimum acreage shall be sufficient to temporarily store 20% of volume of debris accumulated in the 100-yr-frequency design event. Surface acreage shall be calculated assuming a stockpile of 10 feet high with 2:1 fill slopes. A minimum 15-foot wide access road with a 35-foot wide turning radius shall be provided to accommodate equipment access. In addition, a 70-foot long by 15-foot wide strip is required for equipment loading and unloading within an area of sufficient size to maneuver heavy construction equipment.

Minimum Basin Floor Surface Area – Basin floors 1,400 square or greater must be provided with a minimum width of 30 feet.

Outlet Structure – A tower-type outlet is not permitted. Use outlet structure design similar to that used in designs for Tahquitz Creek and Oak Street Debris Basins (slotted/slanted grate). All structures and ramps to structures shall include safety rails/belly bars at all stairways and wherever appropriate. A minimum of two (2) visible depth (paddle) gauges shall be provided.

APPENDIX D

BMP Pollutant Removal Effectiveness

APPENDIX D

BMP POLLUTANT REMOVAL EFFECTIVENESS

BMP Pollutant Removal Effectiveness[1]

Pollutant of Concern	Harvest and Use [9]	Infiltration BMPs[3]	Bioretention	Biofiltration with Partial Infiltration	Biofiltration with No Infiltration	Extended Detention Basins [2]	Sand Filter Basin[8]
Sediment	H	H	H	H	H	M	H
Nutrients	H	H	H	H/M[5]	M/L[6]	M/L[4]	M
Trash	H	H	H	H	H	H	H
Metals	H	H	H	H	H	M	M[7]
Bacteria	H	H	H	H	M	L	M
Oil & Grease	H	H	H	H	H	M	H
Organic Compounds	H	H	H	M	M	L	H
Pesticides and Herbicides	H	H	H	M	M	L	M

Abbreviations:

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

Notes:

(1) Periodic performance assessment and updating of this table has occurred based on updated information from studies from the District, CASQA, Caltrans, the International BMP Database, and others. These effectiveness ratings are bases on the specific BMP designs incorporated into this manual. Effectiveness ratings assume operation of a given BMP in isolation. If BMPs are used in series the overall pollutant removal effectiveness may be increased. Where direct data are not available to describe the performance rating of a certain BMP/pollutant combination, professional judgement was applied based on evaluation of unit operations and processes of BMPs and the associated unit operations and processes that are effective for pollutant removal.

(2) Effectiveness based upon total 72-hour drawdown time.

(3) Includes infiltration basins, infiltration trenches, and permeable pavements without underdrains.

(4) Medium for Phosphorous, Low for Nitrogen.

(5) Nutrient removal is High if Bioretention Soil Media is formulated according to requirements in Fact Sheet 3.8 Bioretention Soil Media. Otherwise nutrient removal efficiency is Medium.

(6) Nutrient removal efficiency is Medium if Bioretention Soil Media is formulated according to requirements in Fact Sheet 3.8 Bioretention Soil Media. Otherwise nutrient removal efficiency is Low. Medium if the standard Bioretention Soil Media is used. If a nutrient sensitive Bioretention Soil Media is used, removal efficiency is High.

(7) High if specialized media targeting metals is used.

(8) Considered to be a Treatment Control BMP. See the WQMP to determine if this BMP can be used.

(9) Cisterns, when associated with an adequate and reliable (year-round) demand for non-potable use of captured storm water (see the applicable WQMP for any specific requirements), have a High effectiveness at removing all pollutants from stormwater runoff. If there is inadequate demand to reliably drain the cistern through non-potable use throughout the year, pollutant removal effectiveness will be low.

References:

Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (WQMPs) in South Orange County. (2017)

International Stormwater Best Management Practices (BMP) Database 2014 Performance Summaries. http://www.bmpdatabase.org/Docs/2014%20Water%20Quality%20Analysis%20Addendum/BMP%20Database%20Categorical_StatisticalSummaryReport_December2014.pdf

International Stormwater Best Management Practices (BMP) Database 2016 Performance Summaries. http://www.bmpdatabase.org/Docs/03-SW-1COh%20BMP%20Database%202016%20Summary%20Stats.pdf

Strecker, E.W., W.C Huber, J.P. Heaney, D. Bodine, J.J. Sansalone, M.M. Quigley, D. Pankani, M. Leisenring, and P. Thayumanavan, “Critical Assessment of Stormwater Treatment and Control Selection Issues.” Water Environment Research Federation, Report No. 02-SW-1. ISBN 1-84339-741-2. 290pp

Oil and grease, Organics, and Trash and Debris based on review of unit operations and processes; comprehensive dataset not generally available. BMP must include design elements to address pollutants of concern.

APPENDIX E

Worksheets for calculating V_{BMP} and Q_{BMP}

Santa Margarita Watershed

V_{BMP} and Q_{BMP} worksheets

These worksheets are to be used to determine the required

Design Capture Volume (V_{BMP})
or the
Design Flow Rate (Q_{BMP})

for BMPs in the Santa Margarita Watershed

To verify which watershed your project is located within, visit

www.rcflood.org/npdes

and use the 'Locate my Watershed' tool

If your project is not located in the Santa Margarita Watershed,

Do not use these worksheets! Instead visit

www.rcflood.org/npdes/developers.aspx

To access worksheets applicable to your watershed

**Use the tabs across the bottom
to access the worksheets for the Santa Margarita Watershed**

Santa Margarita Watershed

BMP Design Volume, V_{BMP} (Rev. 03-2012)

Legend: Required Entries / Calculated Cells

(Note this worksheet shall **only** be used in conjunction with BMP designs from the **LID BMP Design Handbook**)

Company Name

Date

Designed by

County/City Case No

Company Project Number/Name

Drainage Area Number/Name

Enter the Area Tributary to this Feature $A_T =$ acres

85^{th} Percentile, 24-hour Rainfall Depth, from the Isohyetal Map in Handbook Appendix E

Site Location

Township

Range

Section

Enter the 85^{th} Percentile, 24-hour Rainfall Depth $D_{85} =$

Determine the Effective Impervious Fraction

Type of post-development surface cover (use pull down menu) Roofs

Effective Impervious Fraction $I_f =$

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

Use the following equation based on the WEF/ASCE Method

$C = 0.858I_f^3 - 0.78I_f^2 + 0.774I_f + 0.04$ $C =$

Determine Design Storage Volume, V_{BMP}

Calculate V_U, the 85% Unit Storage Volume $V_U = D_{85} \times C$ $V_u =$ (in*ac)/ac

Calculate the design storage volume of the BMP, V_{BMP}.

$$V_{BMP}\ (ft^3) = \frac{V_U\ (\text{in-ac/ac}) \times A_T\ (ac) \times 43{,}560\ (ft^2/ac)}{12\ (in/ft)}$$

$V_{BMP} =$ ft^3

Notes:

Santa Margarita Watershed

BMP Design Flow Rate, Q_{BMP} (Rev. 03-2012)

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_T = acres

Determine the Effective Impervious Fraction

Type of post-development surface cover (use pull down menu): Roofs

Effective Impervious Fraction I_f =

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

Use the following equation based on the WEF/ASCE Method

$C = 0.858I_f^3 - 0.78I_f^2 + 0.774I_f + 0.04$ C =

BMP Design Flow Rate

$Q_{BMP} = C \times I \times A_T$ Q_{BMP} = ft^3/s

Notes:

Effective Impervious Fraction

Developed Cover Types	Effective Impervious Fraction
Roofs	1.00
Concrete or Asphalt	1.00
Grouted or Gapless Paving Blocks	1.00
Compacted Soil (e.g. unpaved parking)	0.40
Decomposed Granite	0.40
Permeable Paving Blocks w/ Sand Filled Gap	0.25
Class 2 Base	0.30
Gravel or Class 2 Permeable Base	0.10
Pervious Concrete / Porous Asphalt	0.10
Open and Porous Pavers	0.10
Turf block	0.10
Ornamental Landscaping	0.10
Natural (A Soil)	0.03
Natural (B Soil)	0.15
Natural (C Soil)	0.30
Natural (D Soil)	0.40
Mixed Surface Types	

Use this table to determine the effective impervious fraction for the V_{BMP} and Q_{BMP} calculation sheets