III.6. Technical Basis for Capture Efficiency-based Performance Criterion

The purpose of this section is to provide the technical basis for the capture efficiency-based expression of the DCV used in throughout the Technical Guidance Document (TGD) and the calculation methods described in the sections above.

III.6.1. Introduction

Every stormwater BMP can be conceptualized as having a storage volume and a treatment rate, in various proportions. Both are important in the long-term performance of the BMP under a range of actual storm patterns, depths, and inter-event times. Long-term performance is measured by the operation of a BMP over the course of multiple years, and provides a more complete metric than the performance of a BMP during a single event, which does not take into account antecedent conditions, including multiple storms arriving in short timeframes. A BMP that draws down more quickly would be expected to capture a greater fraction of overall runoff (i.e. long-term runoff) than an identically sized BMP that draws down more slowly. This is because storage is made available more quickly, so subsequent storms are more likely to be captured by the BMP. In contrast a BMP with a long drawdown time would stay mostly full, after initial filling, during throughout periods of sequential storms. The volume in the BMP that draws down more quickly is more "valuable" in terms of long term performance than the volume in the one that draws down more slowly. In the case of flow-based BMPs, the storage volume is typically not substantial, however it is recognized that flow-based BMPs can achieve high long term capture efficiencies by treating stormwater essentially as it arrives. A method is needed to relate the long-term performance of BMPs to their design attributes so that a common grounds for comparison and "addition" of the benefit of different BMPs is possible.

The permit definition of the LID DCV does not specify a drawdown time, therefore the definition is not a complete indicator of a BMP's level of performance. An accompanying performance-based expression of the LID sizing standard is essential to ensure uniformity of performance across a broad range of BMPs and helps prevents LID BMP designs from being used that would not be effective.

III.6.2. Development of Capture Efficiency-based Performance Criterion

An evaluation of the relationships between BMP design parameters and expected long term capture efficiency has been conducted to address the needs identified above. Relationships have been developed through a simplified continuous simulation analysis of precipitation, runoff, and routing, that relate BMP design volume and storage recovery rate (i.e., drawdown time) to an estimated long term level of performance.

Based on these relationships, it has been demonstrated that a BMP sized for the runoff volume from the 85th percentile, 24-hour storm event (i.e., the DCV), which draws down in 48 hours is capable of managing approximately 80 percent of the average annual. There is long precedent for the assumption that BMPs should draw down in approximately 48 hours, and there is also long precedent for 80 percent capture of average annual runoff as approximately the point at which larger BMPs provide decreasing capture efficiency benefit (also known as the "knee of the curve") for BMP sizing. The characteristic shape of the plot of capture efficiency versus storage volume (Figure III.2) illustrates this concept.

As such, this equivalency (between the DCV drawing down in 48 hours and 80 percent capture) has been utilized to fill three needed roles in this TGD: 1) provide a common currency between volume-based BMPs with a wide range of drawdown rates, 2) provide a means of unifying the sizing of volume-based and flow-based BMPs to allow different types of BMPs to be added as part of a treatment train, and 3) allow flexibility in the design of BMPs while ensuring consistent performance.

III.6.3. Modeling Methodology

The USEPA Stormwater Management Model Version 5.0 (SWMM5.0) was used to simulate the long term average capture efficiency for a range of general BMP design configurations over 22 years of historic hourly precipitation records at the CIMIS Irvine weather station (#75). SWMM was selected for this analysis as it is a relatively simple, open source, continuous simulation model that has well-demonstrated capability for simulation of rainfall-runoff processes in urban environments and simulating transient storage mechanisms in BMPs. A relatively simple representation of BMPs was used to develop the general relationships that conceptualized all BMPs with a simple storage volume and treatment rate. While this representation does not account for the nuances of BMP designs, it is appropriate to develop programmatic sizing factors. Assumed SWMM input parameters are provided in Table III.2. Sensitivity analyses demonstrated that the only inputs with significant sensitivity within typical input ranges were the precipitation and ET inputs and the BMP configurations. These were selected to be representative of Orange County, and results are interpreted to allow scaling across the rainfall zones of the County.

Table III.2: SWMM Simulation Input Parameters

SWMM Parameters	Units	Values
Period of Simulation	years	22 yrs (10/01/1987 to 10/01/2009)
Wet time step	seconds	600
Wet/dry time step	seconds	600
Dry time step	seconds	14,400
Precipitation	inches	Hourly precipitation data from CIMIS Irvine Gage (#75) 279 inches total in period of record
Impervious Manning's n		0.012
Hypothetical drainage area	acres	1
Shape		Rectangular, 250 ft flow path length
Impervious fraction modeled		100%
Slope	ft/ft	0.05
Evaporation	inches	Daily ET data from CIMIS Irvine Gage (#75) 1092 inches reference ETo total in period of record
Depression storage, impervious	inches	0.02, based on Table 5-14 in SWMM manual (James and James, 2000)
Runoff coefficient used to convert precipitation depth to design volume	unitless	0.90
Design capture storm depth (85th percentile, 24-hour depth) calculated from Irvine Gage	inches	0.95
BMP Storage Volume	cu-ft	Varied over continuous range as discrete multipliers on design capture storm depth. Volume at 1.0 × DCV = 0.95 inches × 0.9 × 43,560 sq-ft × (1 ft/12 inches) = 3,100 cu-ft
Drawdown Rate	cfs	Varied over continuous range to represent discrete drawdown times. Q (cfs) = V(cu-ft) / Drawdown time (s) Drawdown rate @ 1.0 × DCV @ 48 hour drawdown time = 3,100 cu-ft / (48 hr × 3600 s/hr) = 0.018 cfs

III.6.4. Detailed Results and Findings

The resulting average annual capture efficiency (i.e., the fraction of average annual runoff that is captured and not immediately bypassed by the BMP) was extracted from model results for each model. The assumed impervious fraction of 100 percent is not important for this analysis because both runoff volume and modeled BMP volume have approximately linear dependency on impervious fraction.

Because this analysis was done at one location in the County, a method is needed to scale these results to different precipitation zones. Areas with larger design capture storm depths (85th percentile, 24-hour depth) should theoretically require larger BMPs for an identical configuration of tributary area and drawdown time. An analysis of several gages in Southern

California has shown that normalizing input scenarios as a fraction of the design capture storm depth allows reliable extrapolation of results throughout the region. These relationships are represented by the nomograph shown as Figure III.2. Functionally, what these relationships show is that for drawdown times larger than 48 hours, a design volume greater than the DCV is needed to achieve 80 percent capture, while for drawdown times less than 48 hours, a design volume less than the DCV can be used to achieve 80 percent capture.

An analogous analysis was conducted for systems with irrigation demand by normalizing input scenarios to fractions of the design capture storm depth and the effective irrigation area to tributary area ratio (EIATA). This analysis considered irrigation demand to be controlled by the area irrigated, landscape demand of this area (i.e., fraction of ETo required for plant use) and the daily ETo timeseries. It was assumed that irrigation would not occur following rainfall until the ET had either summed to a depth equivalent to the rainfall depth or had exceeded 0.25 inches (smaller of these two). Performance relationships are shown in Figure III.3.

III.6.5. Development of Flow-based BMP Capture Efficiency Nomographs

Flow-based BMPs do not have substantial storage volume; therefore function by treating runoff at the rate which it occurs. The concept of a uniform design intensity is commonly used for sizing criteria of flow-based BMPs. This design intensity is appropriately tied to the time of concentration (T_c) of the tributary area, where larger tributary areas should have a lower design intensity because greater attenuation of event peaks is provided in the watershed and the BMP sees lower peaks. While simplified, it can be conceptualized that the T_c of a watershed is the averaging period within which peaks should be averaged.

Because most urban watersheds have T_c much less than 1 hour, hourly precipitation data are not adequate to develop relationships between T_c and the required design intensity to manage a certain percentage of average annual runoff volume. Therefore, 10 years of 5-minute, 0.01" resolution precipitation data were obtained from the Automated Surface Observation System (ASOS) gage at Los Angeles International Airport and used for this analysis.

To represent different increments of T_c, different averaging periods were applied. The resulting intensities were then compared to a range of design intensities to determine the fraction of average annual runoff that intensity would be capable of addressing. It was assumed that if the measured intensity was less than the design intensity, that volume would be fully treated, and if the measured intensity was greater than the design intensity, the volume up to the design intensity would be treated. This implicitly assumes that BMPs are designed to be off-line and maintain their treatment processes even during peak flows.

Figure III.4 presents average annual capture efficiency results for a variety of design storm intensities and drainage area times of concentration.

III.6.6. Note on Using Nomographs to Combine BMPs in Series

The nomographs presented in Figure III.2, Figure III.3, Figure III.4 each show declining response of capture efficiency with design volume and intensity. For example, from Figure III.2, approximately 25% of the DCV is required to achieve the first 40 percent capture of average annual runoff volume, while the remaining 75 percent of the DCV is required to achieve the remaining 40 percent. As such, when combining BMPs in series, capture efficiencies are not directly additive. In order to add the combined effects of BMPs in series, the nomographs should be used by starting at the point on the chart corresponding to the capture efficiency already achieved in upstream BMPs, and moving to the right on the chart along the line corresponding to the drawdown time of the current BMP of interest. This ensures that the appropriate portion of the volume-capture response curve is used.

APPENDIX IV. APPROVED METHODS FOR QUANTIFYING HYDROLOGIC CONDITIONS OF CONCERN (NORTH ORANGE COUNTY)

Hydromodification design criteria for the North Orange County permit area are based on the 2-yr, 24-hr storm event runoff volume, time of concentration, and peak flowrate. Hydrologic analysis of the 2-year, 24-hour storm shall be conducted using the methods described in this section. These include:

- The methods described in the **Orange County Hydrology Manual** (OCEMA 1986).
- The methods described in **Technical Release 55 (TR-55):** *Urban Hydrology for Small Watersheds* (NRCS 1986). TR-55 has the capacity to model watersheds with drainage areas ranging from 0.01 acre (although results from catchments less than 1 acre should be carefully examined) to 25 square miles and time of concentrations ranging from 6 minutes to 10 hours (NRCS 2009).

Priority Projects have the option to either perform the hydrologic calculations using computer simulations or hand calculations. If the **Orange County Hydrology Manual** method is used, the Watershed Modeling System (WMS) software with the Orange County Rational Method interface or hand calculations should be used, consistent with the **Orange County Hydrology Manual**. If the TR-55 method is used, then either the WinTR-55[10] or HEC-HMS[11] programs are appropriate or hand calculations should be consistent with the **TR-55 manual** (NRCS, 1986).

Advantages of using computer simulations is that the runoff hydrograph can be produced with relative ease, which is ideal when simulating post-project drainage conditions which route runoff through detention BMPs. Routing a hydrograph through a BMP is more arduous and time consuming if calculated by hand.

An advantage of WMS with the Orange County Rational Method interface is that it is often used for generating design flows of less frequent design storm events (i.e., 10-year, 25-year, or 100-year) required of flood control analyses, so the same WMS model could be used for both the flood and hydromodification control analyses. It is important to note that WMS is not a

[10] Free WinTR-55 software can be downloaded at: http://www.wsi.nrcs.usda.gov/products/w2q/h&h/tools_models/wintr55.html

[11] Free HEC-HMS software can be downloaded at: http://www.hec.usace.army.mil/software/hec-hms/download.html Loss parameters shall be set to the SCS Curve Number method, transform parameters must be set to the SCS Unit Hydrograph method, and reach routing parameters must be set to the Muskingum-Cunge method.

continuous simulation hydrologic model, and thus cannot be used to meet the South Orange County permit area hydromodification control criteria.

IV.1. Hydrologic Method for 2-year Runoff Volume and Peak

IV.1.1. Storm Depth and Distribution

The 2-yr, 24-hour precipitation depths specified in the Orange County Hydrology Manual shall be used for hydrologic analysis of the 2-year, 24-hour storm.

- For drainage areas below 2,000 feet in elevation a 2.05 storm depth shall be used.
- For drainage areas above 2,000 feet in elevation a 3.81 storm depth shall be used.
- If the Orange County Hydrology Manual is updated over the life of this TGD, the updated 2-year, 24-hour storm depths contained in the updated *Manual* shall supersede these depths.

When using the TR-55 method to produce a hydrograph, the user shall select the Type I rainfall distribution. When using the Orange County Hydrology Manual method, rainfall distribution is imbedded in the WMS-Orange County interface and is provided in the Orange County Hydrology Manual in Section B.

IV.1.2. Runoff Volume

If calculations are performed by hand, the runoff volumes in the existing and proposed conditions shall be calculated using Section C of the Orange County Hydrology Manual or Chapter 2 of the TR-55 manual, which have the same basic methodology. Where inconsistencies (e.g., selection of curve numbers) exist between the two documents, the Orange County Hydrology Manual shall take precedence. For projects less than 5 acres, the difference between runoff volumes in existing and proposed conditions may optionally be calculated using the simple runoff coefficient method (**Appendix III.1.1**). This method tends to under-predict runoff that would occur from pervious areas during a relatively large design storm (pervious runoff coefficient = 0.15) and is likely fairly accurate for runoff from impervious areas (impervious runoff coefficient = 0.90). Therefore, this method tends to result in a larger difference between existing and post-developed runoff coefficient than would be calculated using a more detailed hydrologic analysis and is therefore acceptable where the project proponent elects not to conduct a more detailed hydrologic analysis.

If runoff calculations are performed with modeling software, the runoff volume shall be taken as an output of the WMS-Orange County, WinTR-55, or HEC-HMS models. Input selection for these models shall be consistent with the recommendations found Section C of the Orange County Hydrology Manual or the WinTR-55 Users Guide. Where inconsistencies (e.g., selection of curve numbers) exist between the two documents, the Orange County Hydrology Manual shall take precedence.

When evaluating the effect of retention BMPs on proposed condition runoff volume, volume reduction shall be calculated as the volume that is infiltrated, evapotranspired, or used (i.e., drawn down) over a period of 48 hours, starting at the BMP brim full capacity. Volume treated and discharged to surface water shall not be considered in this calculation. The volume reduction shall not be greater than the total retention volume in the BMP.

IV.1.3. Peak Runoff Flowrate

Peak runoff flowrate shall be calculated using one of the following methods depending on watershed size:

The Rational Method described in Section D of the **Orange County Hydrology Manual** shall be used for drainage areas less than 1 square mile (640 acres). For redevelopment projects less than 5 acres, the simplified runoff coefficient method described in **Appendix III.1.2** can be used to compute the runoff coefficient for rational method calculations.

The Unit Hydrograph Method described in Section E of the **Orange County Hydrology Manual** shall be used for drainage areas greater than or equal to 1 square mile.

Alternatively, peak flowrate shall be calculated using the Graphical Peak Discharge Method described in Chapter 4 of the **TR-55 manual** or the Tabular Hydrograph Method described in Chapter 5 of the same document. When evaluating the effect of BMPs on the proposed condition peak runoff flowrate, the effect of the BMP should be estimated using one of the aforementioned modeling programs because hand calculations are not ideal for the routing analyses required.

Example IV.1 provides an example runoff volume and peak flow calculation for a simple project using WinTR-55. This example is not intended to be exhaustive of the methods that could be used to calculate runoff volume and peak flow.

IV.2. Hydrologic Method for Time of Concentration

Time of concentration (T_c) shall be calculated using one of the following approved methods:

If computing by hand, the methods described in Section D of the **Orange County Hydrology Manual** or the **TR-55 manual** shall be used. The Orange County method entails summing the initial time of concentration, based on a nomograph, with the subsequent time it takes to pass flow through downstream conveyances. The TR-55 method sums the travel times for sheet flow, shallow concentrated flow, and channel flow for a given flow path.

If using a modeling tool, the WinTR-55 model is the only tool that provides an acceptable model-calculated method of calculating Tc through its Time of Concentration Details window. The inputs provided to this window shall be per guidance contained in the **Orange County Hydrology Manual** or the **TR-55 manual** and shall be submitted with the Project WQMP documentation.

WMS-Orange County will help the user estimate the Tc of a subarea when using the GIS interface or it can be entered manually. HEC-HMS does not assist the user in estimating Tc and its transform input parameter is actually lag time, which is 0.6 times the Tc, according to an empirical relationship developed by the Natural Resource Conservation Service (NRCS). The use of these models must be supported by hand calculations of T_c per criteria above.

When evaluating the effect of storage and treatment BMPs on the proposed condition time of concentration, the BMP lag component of T_c shall be estimated as the time required for the BMP to being discharging to the downstream receiving water during the design storm simulation. This can be calculated by (1) determining the volume the BMP can receive before it begins to discharge, (2) plotting the post-developed runoff hydrograph for the 2-year, 24-hour storm event, and (3) by determining the time on the hydrograph at which the cumulate volume exceeds the volume calculated in step 1.

Example IV.1 provides an example time of concentration calculation for a simple project using the T_c window in WinTR-55. This example is not intended to be exhaustive of the methods that could be used to calculate T_c.

IV.3. Hydrologic Calculation Examples with WinTR-55

Example IV.1: Computing Volume and Peak Flowrate Using WinTR-55

Given:

- Project Elevation: 1,200 ft
- Drainage Area = 2.0 acres
- Hydrologic Soil Group = B
- Existing Condition: 1.8 acres of herbaceous grassland in fair condition, with 0.2 acres of miscellaneous roads and structures; imperviousness = 11 percent
- Existing flow path: 100 ft overland sheet flow @ 3% slope, 50 ft shallow concentrated flow @ 3% slope (unpaved), 300 ft ditch @ 0.5% slope
- Proposed Condition: multi-family residential; imperviousness = 80 percent
- Proposed flow path: 100 ft overland sheet flow @ 10% slope (roofs and driveways); 400 ft of stormdrain @ 0.5% slope

Required:

- Calculate runoff volume and peak flowrate in existing and proposed conditions
- Compute BMP volume needed to reduce post-developed runoff volume to within 5% of existing condition runoff volume for the 2-year storm event.

Results:

1) Existing Condition: Peak Flow Rate (cfs) = 0.28, Runoff Volume (cubic feet) = 1,249,

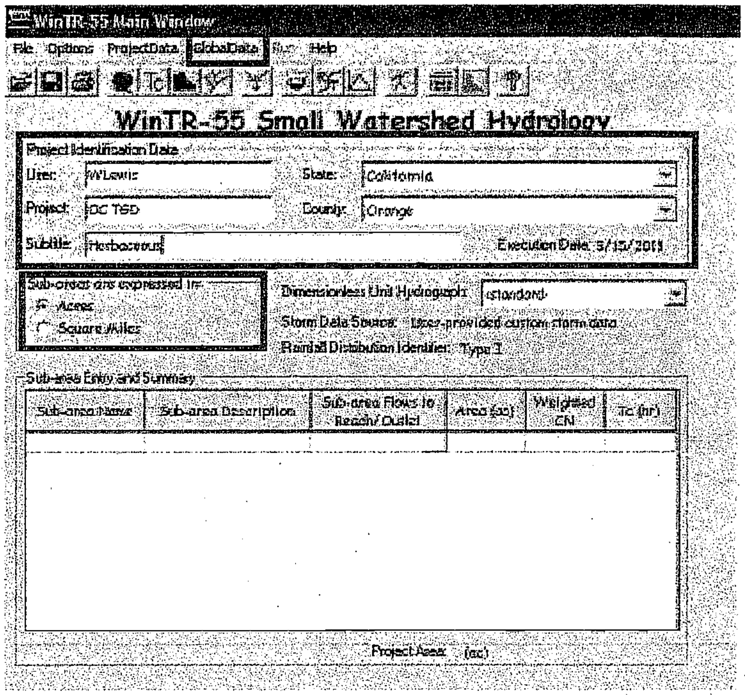
   Proposed Condition: Peak Flow Rate (cfs) = 2.01, Proposed Runoff Volume (cubic feet) = 9,039

2) Required BMP Volume (cubic feet) = (9,039 – (1,249 × 1.05)) = 7,730 cu-ft

Solution Steps:

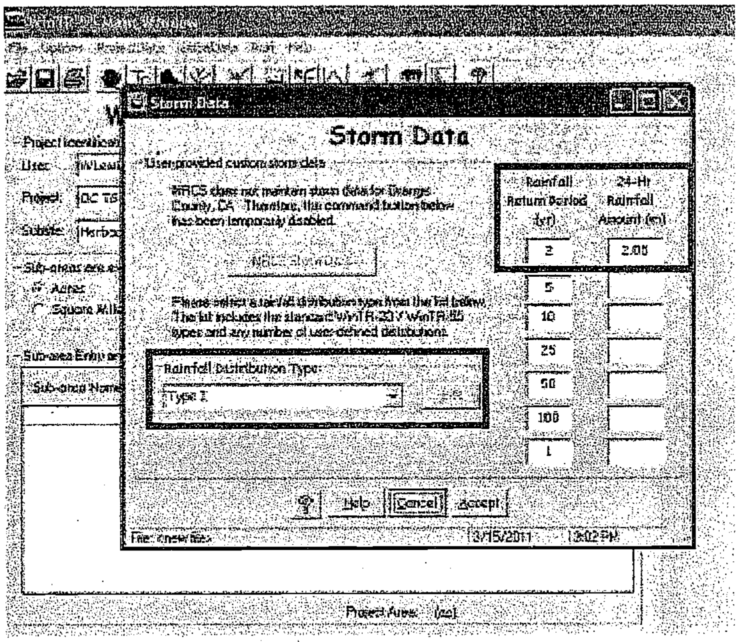
1) Open WinTR-55 and complete the "Project Identification" fields (Figure IV.1).

Figure IV.1: WinTR-55 home screen



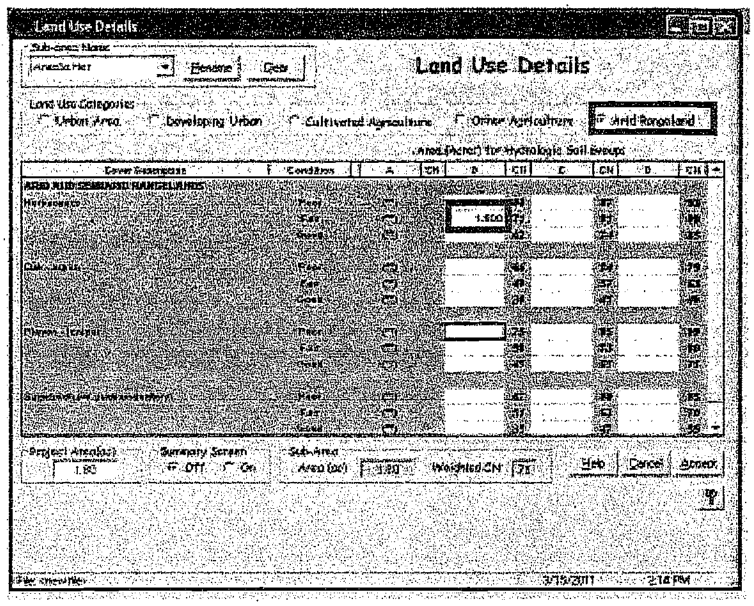
2) Under the "GlobalData" heading select "Storm Data" and select "Type 1" as the rainfall distribution type and enter 2.05" as the 2-year storm event (the project is below an elevation of 2,000 feet. The design storm would be 3.81" if the project was located above 2,000 feet.) (Figure IV.2). Accept these changes and save the project.

Figure IV.2: WinTR-55 Storm Data screen



3) From the home screen, select "Land Use Details" from the "ProjectData" heading, name the sub-area, and select the radio button for "Arid Rangeland" to begin setting up the existing condition. Enter 1.8 acres for "Herbaceous - Fair Condition" under Hydrologic Soil Group B before selecting the "Urban Area" radio button and entering 0.2 acres under "Paved parking lots, roofs, and driveways," again for Hydrologic Soil Group B (Figure IV.3). The program will calculate an area weighted curve number. Accept changes and return to the home screen.

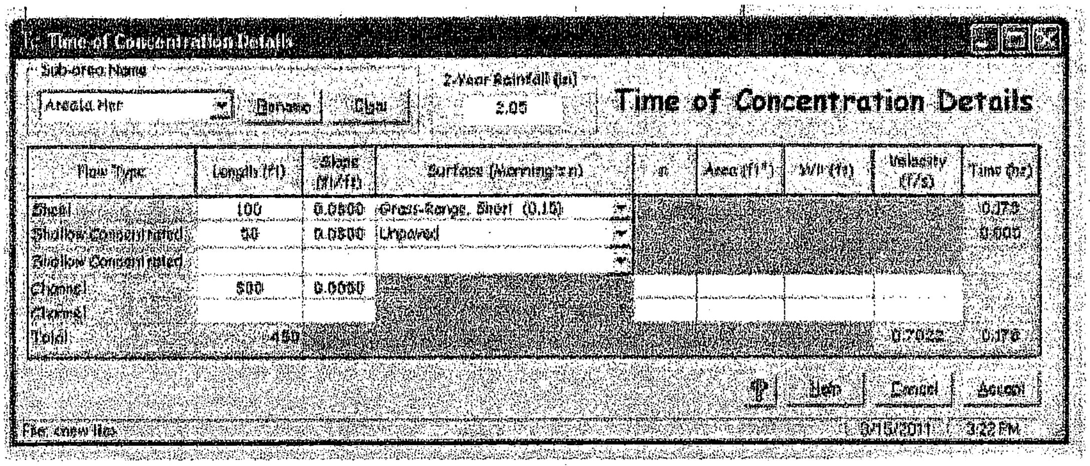
Figure IV.3*: *WinTR-55 Land Use Details screen



4) Select "Outlet" under the "Sub-area Flows to Reach/Outlet" pull-down menu.

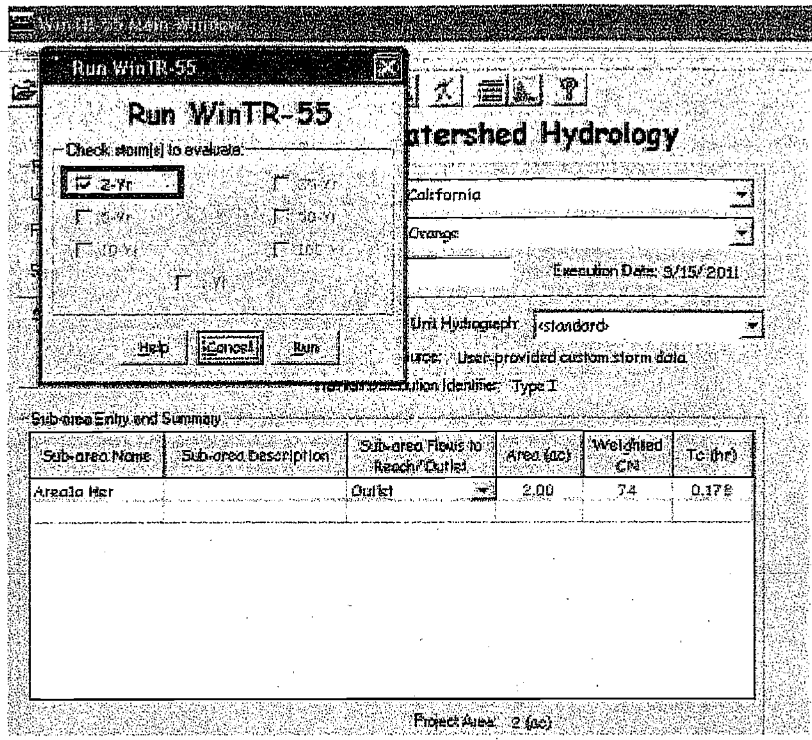
5) Under the "ProjectData" heading select "Time of Concentration Details" and enter lengths, slopes, and Manning's roughness coefficients (if necessary) for relevant flow types (Figure IV.4). Save the project.

Figure IV.4: WinTR-55 Time of Concentration Details screen



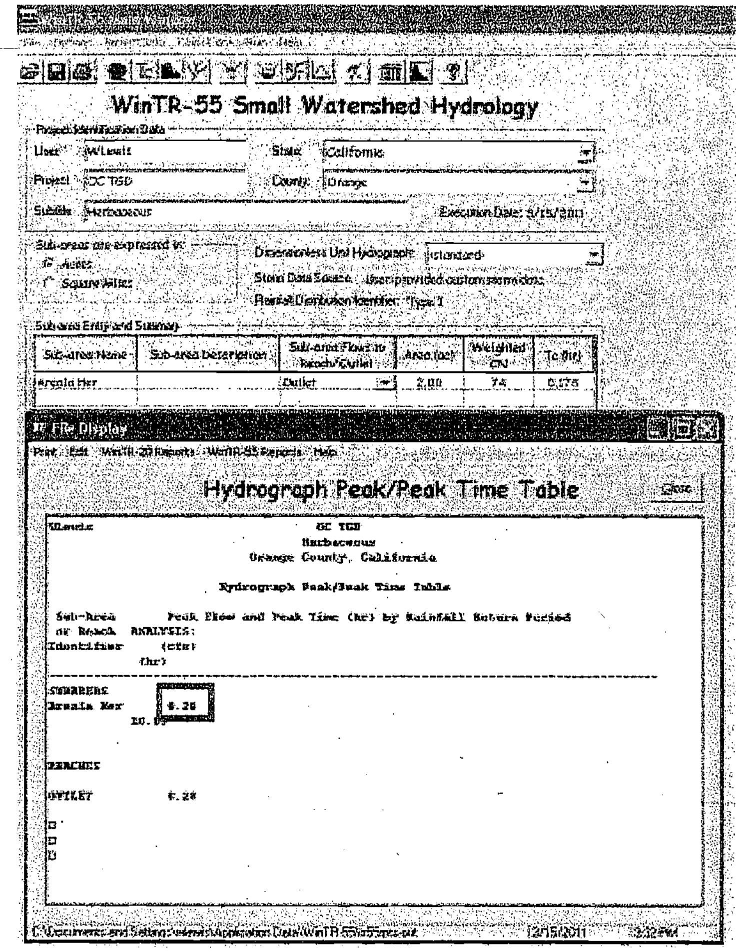
6) Select the "Run" heading and ensure that the 2 year storm box is checked. No other recurrence interval storm depths were entered and are therefore not an option (Figure IV.5).

Figure IV.5*: *WinTR-55 Run Model screen



7) Peak discharge is provided in the "Hydrograph Peak/Peak Time Table" that appears following the completion of the model run. Record the "Peak Discharge (cfs)" (Figure IV.6).

Figure IV.6*: *WinTR-55 Hydrograph Peak/Peak Timetable screen

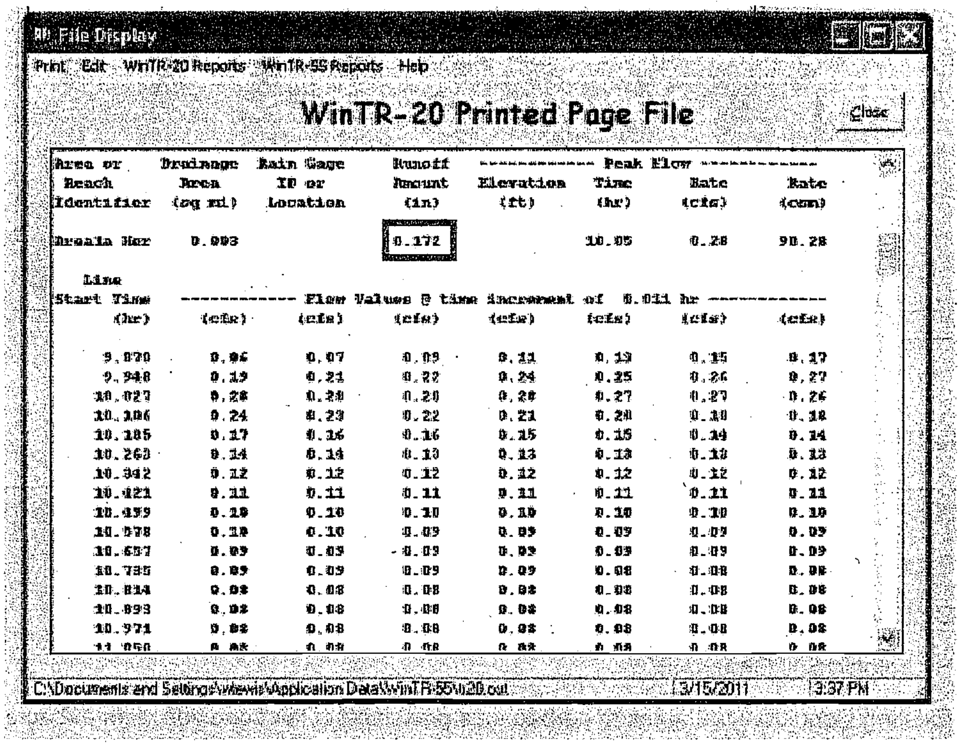


8) Within the "Hydrograph Peak/Peak Time Table" select the WinTR-20 pull-down menu and select "Printed Page File" to access the "WinTR-20 Printed Page File."

9) Scroll down to the page titled TR20.out and record the "Runoff Amount (in)." Convert the rainfall runoff depth into acre feet (dividing by 12 inches/foot and multiplying by the total acreage). Record the total volume of runoff from the modeled area (Figure IV.7).

Existing 2-yr Runoff volume = 0.172 inches × 2 acres × 43,560 sq-ft/ac × 1ft/12inches = 1,249 cu-ft

***Figure IV.7**: WinTR-20 Printed Page File screen*



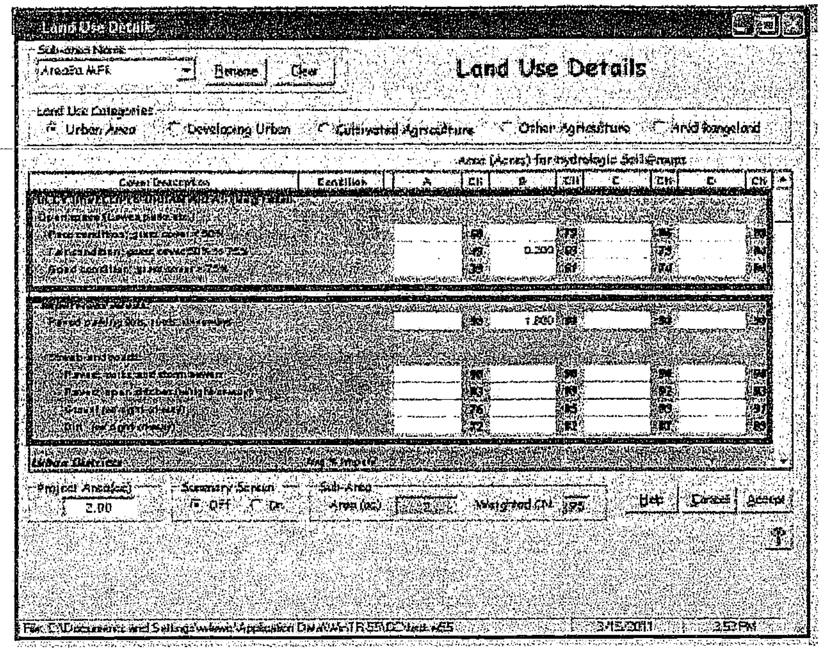
Area or Reach Identifier	Drainage Area (sq mi)	Rain Gage ID or Location	Runoff Amount (in)	Peak Flow Elevation (ft)	Peak Flow Time (hr)	Peak Flow Rate (cfs)	Peak Flow Rate (csm)
[illegible]	0.003		0.172		10.05	0.28	90.28

Line Start Time (hr)	Flow Values @ time increment of 0.011 hr (cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)
9.870	0.06	0.07	0.09	0.11	0.13	0.15	0.17
9.948	0.19	0.21	0.22	0.24	0.25	0.26	0.27
10.027	0.28	0.28	0.28	0.28	0.27	0.27	0.26
10.106	0.24	0.23	0.22	0.21	0.20	0.18	0.18
10.185	0.17	0.16	0.16	0.15	0.15	0.14	0.14
10.263	0.14	0.14	0.13	0.13	0.13	0.13	0.13
10.342	0.12	0.12	0.12	0.12	0.12	0.12	0.12
10.421	0.11	0.11	0.11	0.11	0.11	0.11	0.11
10.499	0.10	0.10	0.10	0.10	0.10	0.10	0.10
10.578	0.10	0.10	0.09	0.09	0.09	0.09	0.09
10.657	0.09	0.09	0.09	0.09	0.09	0.09	0.09
10.735	0.09	0.09	0.09	0.09	0.08	0.08	0.08
10.814	0.08	0.08	0.08	0.08	0.08	0.08	0.08
10.893	0.08	0.08	0.08	0.08	0.08	0.08	0.08
10.971	0.08	0.08	0.08	0.08	0.08	0.08	0.08
11.050	0.08	0.08	0.08	0.08	0.08	0.08	0.08

10) From the same "WinTR-20 Printed Page File" select the time and rate of runoff values for the duration reported and transfer these values into a plotting program (i.e. Microsoft Excel®) (Figure IV.7). Save Project, WinTR-20, and WinTR-55 outputs as records.

11) Initiate a second WinTR-55 Project and complete steps 1 through 11 for the proposed scenario. Selection of land uses for the proposed condition shall be limited to options under the headings of "Fully Developed Urban Areas (Veg Estab.)" and "Impervious Area" (Figure IV.8). Selected land uses should reflect the proposed percent impervious (i.e. 80% impervious would be represented by selecting 80% "Paved parking lots, roads, driveways" and 20% for the appropriate pervious condition by area).

Figure IV.8*: *WinTR-55 Proposed Condition Land Use Details screen



Example IV.2: Computing Time of Concentration using TR-55 Methods

Given:

1) Project Elevation: 1,200 ft
2) Drainage Area = 2.0 acres
3) Hydrologic Soil Group = B
4) Existing Condition: 1.8 acres of herbaceous grassland in fair condition, with 0.2 acres of miscellaneous roads and structures; imperviousness = 11 percent
5) Existing flow path: 100 ft overland sheet flow @ 3% slope; 50 ft shallow concentrated flow @ 3% slope (unpaved), 300 ft ditch @ 0.5% slope
6) Proposed Condition: multi-family residential; imperviousness = 80 percent
7) Proposed flow path: 100 ft overland sheet flow @ 10% slope (roofs and driveways); 400 ft of stormdrain @ 0.5% slope
8) Infiltration basin proposed for project with retention storage capacity of 7,730 cu-ft (See Example IV.1)

Required:

a. Calculate T_c of existing condition
b. Calculate T_c of proposed condition without BMPs

c. Calculate effective T_c of proposed condition with BMPs

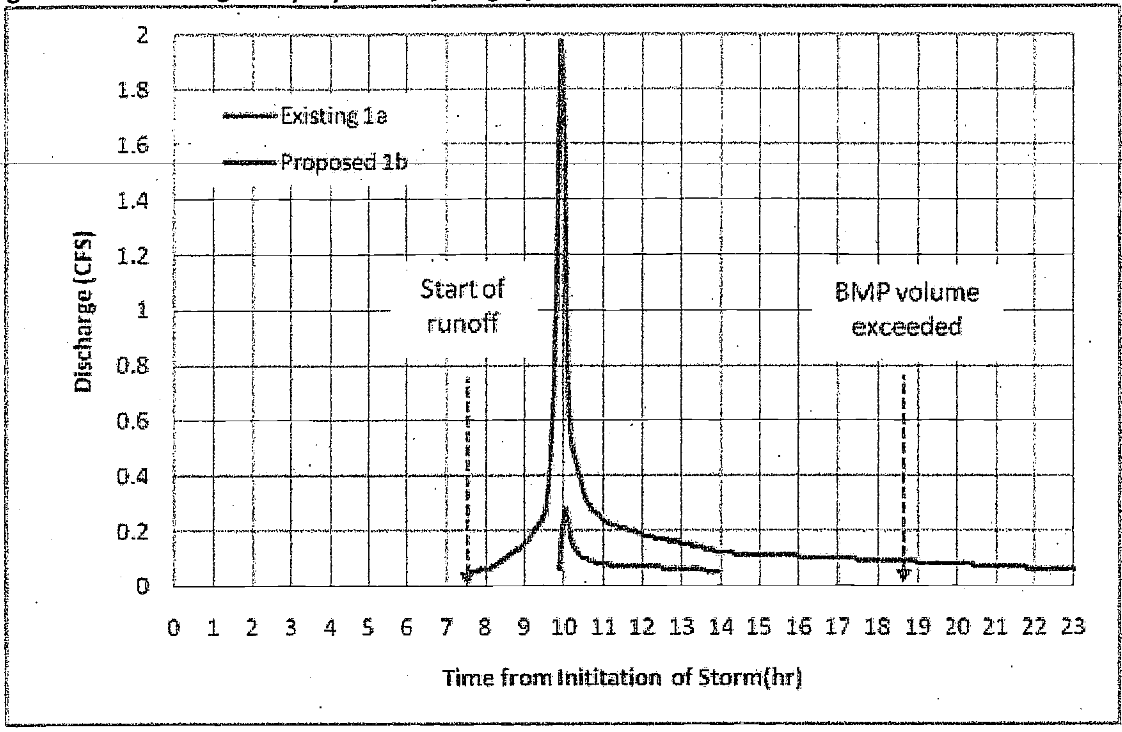
Solution:

1) 0.178 hr
2) 0.013 hr (0.1 used by TR-55 as a minimum value)
3) 9.94 hr

Solution Steps:

1) See Example IV.1 Steps 1 through 12 for direction in setting up existing and proposed WinTR-55 models, recording relevant information, and obtaining data to plot hydrographs.
2) Times of Concentration for existing conditions and proposed conditions without BMPs can be taken directly from the WinTR-55 Tc model screen.
3) The time of concentration of the proposed condition with BMPs can be estimated as difference between the point of the storm event where runoff begins and the point in the storm event at which the runoff volume exceeds the BMP volume and discharge would be expected to occur. The timeseries output from the TR-20 window can be plotted in a spreadsheet program. Based on this example, runoff begins 7.6 hours and the runoff volume exceeds the BMP volume (7,730 cu-ft) at 18.6 hours. Therefore the effective time of concentration with the BMP included is approximately 11 acres. This is clearly not a concern and more detailed assessment of T_c is not required.

Figure IV.9: Existing and proposed hydrographs



APPENDIX V. APPROVED METHODS FOR QUANTIFYING HYDROLOGIC CONDITIONS OF CONCERN (SOUTH ORANGE COUNTY)

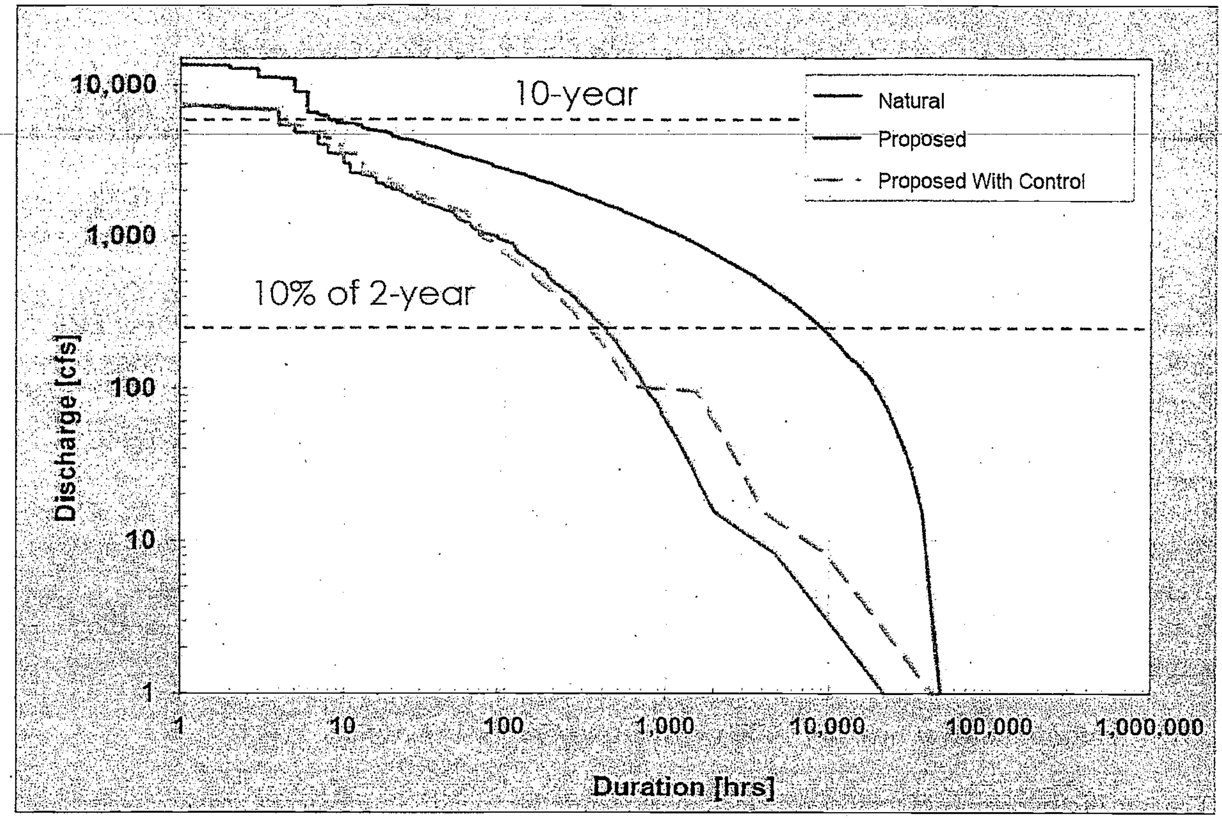
If a HCOC exists, projects in the South Orange County permit area shall use an approved continuous simulation model such as EPA Stormwater Management Model (SWMM) or EPA Hydrologic Simulation Program – FORTRAN (HSPF), to evaluate compliance with the flow-duration-based performance criteria of the interim hydromodification standard. The following sections describe design references that have been prepared to streamline and guide these calculations.

The final hydromodification standard requires the preparation of a hydromodification management plan (HMP), which will prescribe the hydrologic analysis methods and performance criteria that will apply. When the SOC HMP is adopted, it will supersede the requirements of this section to the extent that it is applicable.

V.1. Hydromodification Control Flow Duration Control Analysis

The interim hydromodification standard in the South Orange County permit area focuses on controlling hydromodification by mimicking pre-development (naturally occurring) flow magnitudes and durations over a long period of record rather than for the discrete 2-year storm event. A flow duration curve is the primary means of demonstrating changes in flow magnitudes and durations over a continuous period of record. A flow-duration curve is a plot of discharge versus the duration of time the discharge is exceeded. It is developed through continuous simulation of project under the following conditions: pre-developed (natural), post-developed, and post-developed with controls. An example flow duration curve is show in **Figure V.1**.

Figure V.1. Example Flow Duration Chart



In order to mitigate HCOCs in the South Orange County permit area, flow rates and durations must be controlled between 10 percent of the 2-year storm event and the 10-year storm event, as indicated by purple dashed lines on **Figure V.1**. This means that the post-development flow duration curve (red line in **Figure V.1**) needs to be lowered such that it is at or below the pre-development flow duration curve (green line) within the bounds of the purple dashed lines. In order to accomplish this, site design, volume reduction, and flow duration control BMPs can be used. This process must be based on continuous simulation of stormwater controls or through use of design charts developed from continuous simulation of stormwater controls.

V.2. South Orange County Interim Hydromodification Sizing Tool

Orange County Public Works has prepared the South Orange County Interim Hydromodification Sizing Tool to assist preparers with sizing of BMPs to comply with the SOC interim hydromodification sizing standard. This tool is based on nomographs for a range of BMPs developed through continuous simulation in EPA SWMM5.0. The sizing tool (Excel spreadsheet) and accompanying memorandum are available for download at: http://www.ocplanning.net/WaterQuality.aspx.

V.3. Guidelines for Project-Specific Flow Duration Analysis

This section describes the methods that shall be used by applicants wishing to perform a project-specific analysis for compliance with the SOC interim hydromodification standard instead of using the tool described in Section V.2. This section also provides documentation of the assumptions that were used to develop the interim sizing tool to provide a reference point for Project WQMP preparers and reviewers.

(Placeholder for work in progress)

APPENDIX VI. APPROVED METHODS FOR CALCULATING ALTERNATIVE COMPLIANCE VOLUME FOR LID

This appendix contains technical guidance for calculating the alternative compliance volume for projects that do not fully address LID performance standard through one of the primary pathways. This section is intended to be used as referenced from **Section 2.4 of the Model WQMP**. For the purposes of developing an alternative compliance program, the remaining ("unmet") portion of the DCV is also termed the *alternative compliance volume*. This volume is determined based on the difference between the target 80 capture efficiency and the capture efficiency achieved by the LID BMPs that are provided for the project before entering the alternative program. The alternative compliance volume is first calculated before the application of water quality credits, and then water quality credits are used to reduce this volume to the alternative compliance volume.

VI.1. Calculating Alternative Compliance Volume without Water Quality Credits

This section describes the method for calculating the alternative compliance volume prior to application of water quality credits.

Calculate the capture efficiency achieved upstream of the alternative compliance program. In the North Orange County permit area, this may include the effects of on-site LID BMPs and/or sub-regional/regional LID BMPs. In the South Orange County permit area, this will only include the effects of on-site LID BMPs. Methods of calculating capture efficiency are provided in Section III.4.

Using **Figure VI.1**, find the already-achieved capture efficiency on the horizontal axis and read upward to the line on the chart. Pivot 90 degrees and read to the vertical axis. This is the fraction of the design capture storm depth remaining to be met. Multiply this value by the design capture storm depth for the project (as determined from **Figure III.1**) to determine the remaining storm depth to be managed in the alternative compliance plan.

Compute the volume of runoff from the project for the storm depth calculated in (2), by using the hydrologic methods described in Section III.1.1. This is the remaining volume to be managed (i.e., the *alternative compliance volume*), expressed in cubic feet.

Example VI.1: Calculating Remaining LID Volume for Alternative Compliance

Given:
• 85th percentile, 24-hr storm depth = 0.85 inches (**Figure III.1**)
• Drainage Area = 1.5 acres

- Imperviousness = 80%
- Upstream LID BMPs achieve 60 percent average annual capture efficiency

Required:

- Compute remaining LID volume transferred to alternative program

Solution:

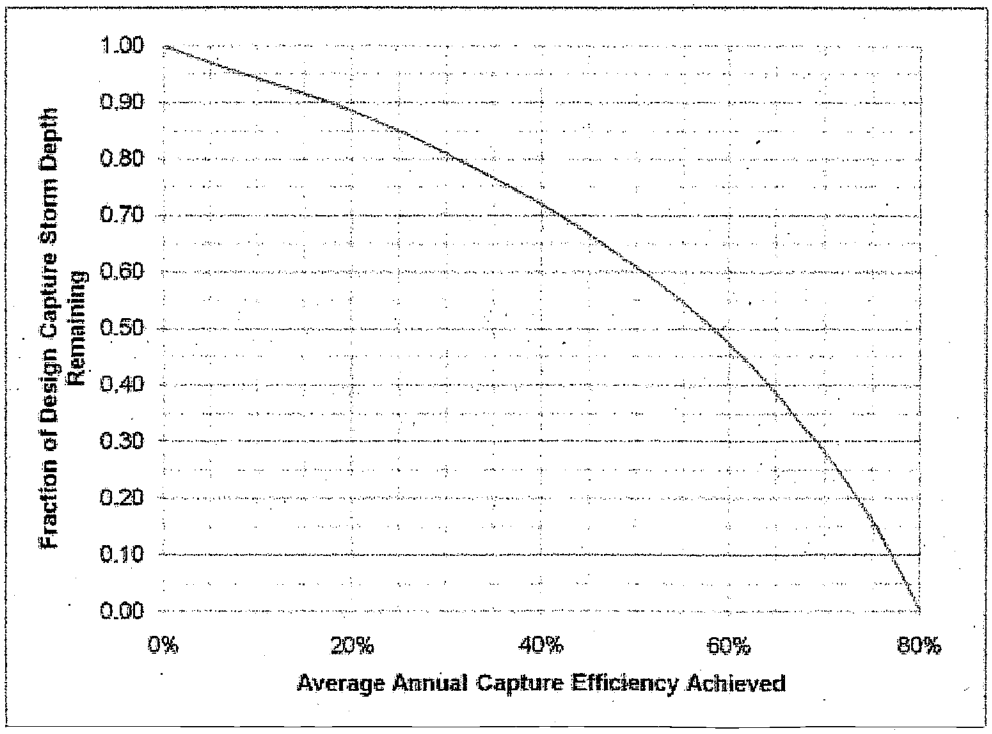
1) Capture efficiency achieved = 60 percent (given)

33) From **Figure VI.1**, the unmet fraction of the design capture storm depth is 0.47. The unmet design storm depth = 0.47 × 0.85 inches (given) = 0.40 inches

34) V_{REMAIN} = 1.5 ac × 0.40 inches × (0.8×0.75 + 0.15) × 43,560 sf/ac × 1/12 in/ft = 1,630 cu-ft

35) This is the volume that must be addressed through alternative compliance programs.

Figure VI.1: Lookup Graph for Fraction of Design Capture Storm Depth Remaining



VI.2. Applying Water Quality Credits to Adjust Alternative Compliance Volume

Water quality credits may be applied to reduce the *alternative compliance volume*. Alternative compliance volume obligations are computed as described in Section VI.1 and expressed in terms of a simple volume. Water quality credits are then computed based on the original DCV for the project and may fully or partially off-set the remaining alternative compliance volume. The volume of alternative compliance obligations offset by Water Quality Credits shall be calculated in one of two ways, as described below. Eligibility of projects to claim water quality credits is described in **Section 3.1 of the Model WQMP**.

VI.2.1. Method 1: Applying Water Quality Credits to Redevelopment Projects Reducing Overall Impervious Footprint

For eligible redevelopment projects that reduce the overall impervious footprint of the project site compared to current use, the volumetric offset provided by water quality credits shall be calculated as follows:

Calculate an equivalent "existing" DCV for the site using the pre-project imperviousness, the design capture storm depth (Figure III.1) and the method described in Section **III.1.1**)
Calculate the DCV for the site under the proposed development plan using the proposed project imperviousness, the design capture storm depth (Figure III.1) and the method described in Section **III.1.1**)
The difference between the volumes calculated in (1) and (2) is equal to the Credit Volume, which may be applied to off-set the alternative compliance volume.

An example of this calculation is provided in Example VI.2.

Example VI.2: Calculating Water Quality Credits for Projects Reducing Imperviousness

Given:

- 85th percentile, 24-hr storm depth = 0.85 inches (Figure III.1)
- Drainage Area = 1.5 acres
- Pre-project Imperviousness = 100%
- Post-project Imperviousness = 70%

Required:

- Compute the water quality credit that could be claimed for reducing project imperviousness

Solution:

1) DCV (pre-project) = 1.5 ac × 0.85 inches × (1.0×0.75 + 0.15) × 43,560 sf/ac × 1/12 in/ft = 4,170 cu-ft
2) DCV (pre-project) = 1.5 ac × 0.85 inches × (0.7×0.75 + 0.15) × 43,560 sf/ac × 1/12 in/ft = 3,120 cu-ft
3) Credit volume = DCV(pre) – DCV(post) = 4,170 cu-ft - 3,120 cu-ft = 1,050 cu-ft
4) This is the credit volume that can be applied to reduce "unmet" volume.

VI.2.2. Method 2: Applying Water Quality Credits to Projects Based on Project Type and Density

Water Quality Credits are expressed in terms of percentages of the original DCV (i.e., the runoff from the design capture storm depth in the proposed condition before applying any BMPs). This section is intended to be applicable for calculating the volume (cu-ft) corresponding to these credits. The applicability of credits is described in **Section 3.1 of the Model WQMP**. The user is expected to enter this section with the total WQ credit percentage.

The volume credit would be calculated as the DCV of the proposed condition multiplied by WQ Credit percentage:

$$\text{Credit Volume} = \text{Original DCV} * \sum \text{Credit Percentages Claimed}$$

An example of this calculation is provided in Example VI.3.

Example VI.3: Applying Water Quality Credits to Reduce Alternative Compliance Volume

Given:

- 85th percentile, 24-hr storm depth = 0.85 inches (Figure III.1)
- Drainage Area = 1.5 acres
- Imperviousness = 80%
- Alternative compliance volume before claiming water quality credits = 1,630 cu-ft
- Total credit based on applicability described in **Section 3.1 of the Model WQMP**: 30 percent

Required:

- Compute remaining unmet volume after applying water quality credits

Solution:

1) Add all applicable credits = 20% + 10% = 30% (per applicability described in **Section 3.1of the Model WQMP**)
2) DCV (unmitigated) = 1.5 ac × 0.85 inches × (0.8×0.75 + 0.15) × 43,560 sf/ac × 1/12 in/ft = 3,470 cu-ft
3) Credit volume = total credit × original DCV = 30% × 3,470 cu-ft = 1,040 cu-ft
4) Remaining volume after credits = 1,630 cu-ft – 1,040 cu-ft = 590 cu-ft
5) This is the remaining volume that must be addressed through other forms of alternative compliance.

VI.3. Stormwater Quality Design Volume/Flow Calculations for Sizing Treatment Control BMPs for Alternative Compliance

The following sections describe how a specified alternative compliance volume (after adjusting for water quality credits) shall be translated to volume-based or flow-based sizing criteria for treatment control BMPs.

VI.3.1.1. Volume-based Treatment Control BMPs

Volume-based treatment control BMPs shall be sized such that they capture and treat the remaining alternative compliance volume.

For example, if as part of an alternative compliance plan, 10,000 cu-ft of remaining volume was designated to be treated by a treatment control BMP, the BMP would be sized with a design volume of 10,000 cu-ft..

VI.3.1.2. Flow-based Treatment Control BMPs

Because unmet volume is expressed in units of volume, this unmet volume must be translated to a flowrate for sizing of flow-based treatment control BMPs. This section describes the method by which an unmet runoff volume would be addressed by a flow-based treatment control BMP. The method requires that the drainage area to the proposed flow-based treatment control BMP be known.

1) For the catchment to which the flow-based BMP will be applied, convert the unmet volume to an unmet storm depth using the method of back-computing storm depth described in Section III.1.1 and Example III.2.
2) Divide the back-computed storm depth by the design capture storm depth to yield the unmet fraction of the design storm depth over the tributary area to the BMP. If this value is greater than 1.0, increase the area tributary to the flow-based BMP.
3) Estimate the time of concentration (T_c) of the catchment.
4) Use
5)
6) Table VI.1to look up the multiplier based on the calculated T_c. Multiply the looked up value by the remaining fraction of the design capture storm depth (Step 2) to yield the design intensity.
7) Use the hydrologic method described in Section III.1.2 to compute the design flow.
8) This method can also be used in reverse if necessary.

Table VI.1: Table of Multipliers for Computing Remaining Design Storm Intensity

Time of Concentration, minutes	Multiplier to Convert Remaining Fraction of Design Capture Storm Depth to Design Intensity, in/hr
60	0.15
30	0.18
20	0.19
15	0.21
10	0.23
5	0.26

Example VI.4: Computing the Required Design Flowrate to Mitigate Remaining Alternative Compliance Volume

Given:

- 85th percentile, 24-hr storm depth = 0.85 inches (Figure III.1)
- Drainage area to proposed flow-based BMP = 1.5 acres
- Imperviousness of drainage area = 80%
- Time of concentration (T_c) of the drainage area = 15 minutes
- Remaining volume (designated to be managed with the proposed BMP) = 1,200 cu-ft

Required:

- Compute required design flowrate to mitigate the alternative compliance volume

Solution:

1) Equivalent storm depth = 1,200 cu-ft × 12 in/ft/[(0.75×0.8+0.15) ×1.5 ac ×43560 sf/ac] = 0.29 inches
2) Fraction of design capture storm depth = 0.29 inches/0.85 inches = 0.35 = 35% of DCV
3) From
4)
5) **Table VI.1**, the multiplier for T_c of 15 minutes is 0.21 in/hr
6) Design intensity equivalent to the remaining unmet volume = 0.21 in/hr × 0.35 = 0.074 in/hr
7) Design flow equivalent to the remaining alternative compliance volume = (0.75×0.8+0.15) × 0.074 in/hr ×1.5 ac = 0.083 cfs
8) This is the design flowrate that must be provided for the 1.5 acre tributary area to address 1,200 cu-ft of remaining volume.

Worksheet G: Alternative Compliance Volume Worksheet

Step 1: Determine the alternative compliance volume without water quality credits				
1	Determine the capture efficiency achieved in upstream BMPs using **Appendix III**, X_1 (%)	X_1=		%
2	Enter design capture storm depth from **Figure III.1**, *d* (inches)	d=		inches
3	Using **Figure VI.1**, pivot from where X_1 intersects the curve to determine the fraction of design capture storm depth remaining to be met, Y_1	Y_1=		
4	Calculate the design depth that must be managed in alternative compliance BMPs, $d_{alternative} = Y_1 \times d$	$d_{alternative}$=		inches
5	Compute the alternative compliance volume corresponding to $d_{alternative}$ using the hydrologic methods described in Section **III.1.1**, *ACV* (cu-ft)	ACV=		cu-ft
Step 2: Determine Credit Volume				
Method 1: Determine Credit Volume based on Reducing Impervious Footprint				
1	Enter design capture storm depth from **Figure III.1**, *d* (inches)	d=		inches
2	Using d, calculate the DCV using the pre-project imperviousness and the methods described in **Appendix III**, DCV_{pre} (cu-ft).	DCV_{pre}=		cu-ft
3	Using d, calculate the DCV using the proposed imperviousness and the methods described in **Appendix III**, DCV_{post} (cu-ft).	DCV_{post}=		cu-ft
4	Calculate the $Credit\ Volume = DCV_{pre} - DCV_{post}$ (cu-ft).	Credit Volume=		cu-ft
Method 2: Determine Credit Volume based on Project Type and Density				
1	Determine the sum of the Credit Percentages applicable to the Project, ∑*Credit Percentages* (%). (See Section 2.4 of the WQMP)	∑*Credit Percentages* =		%
2	Enter design capture storm depth from **Figure III.1**, *d* (inches)	d=		inches
3	Using d, calculate the DCV using the proposed imperviousness without BMPs and the methods described in **Appendix III**, $DCV_{post\ no\ BMP}$ (cu-ft).	$DCV_{post\ no\ BMP}$=		cu-ft
4	Calculate the $Credit\ Volume = DCV_{post\ no\ BMP} \times \sum Credit\ Percentages$	Credit Volume=		cu-ft

Worksheet G: Alternative Compliance Volume Worksheet

Step 3: Determine the Alternative Compliance Volume after WQ Credits				
1	Enter design capture storm depth from **Figure III.1**, *d* (inches)	d=		inches
2	Using d, calculate the DCV using the proposed imperviousness and the methods described in **Appendix III**, DCV_{post} (cu-ft).	DCV_{post}=		cu-ft
3	Calculate the alternative compliance volume, $ACV = DCV_{post} - Credit\ Volume$	ACV=		cu-ft

APPENDIX VII. INFILTRATION RATE EVALUATION PROTOCOL AND FACTOR OF SAFETY RECOMMENDATIONS

VII.1. Introduction

Soil characterization and infiltration testing is required in order to properly size and locate stormwater management facilities. The purpose of this appendix is to provide guidance for investigating infiltration at both the project planning and design phases, as well as provide requirements for applying a factor of safety to testing results.

VII.1.1. Two phases of assessment

The role of soil characterization and infiltration testing differs with the phase of project development as described below.

Site Assessment / Project Planning Phase: Soil characterization or infiltration testing may be conducted to determine if infiltration is a potentially feasible BMP and/or where on the site infiltration is potentially infeasible. The intent of this investigation is to identify if the project site, or a portion of the site, has soils that are clearly unsuitable for infiltration. For those sites or portions of the site where soils are unsuitable, infiltration BMPs can be eliminated from consideration. The intent of this testing is not to prove definitively that infiltration is feasible. Simpler methods may be used to determine infiltration potential at this phase. The observed infiltration rate is adjusted to account for the type of test and the uncertainty of the testing method and reported as the *measured infiltration rate* for the purpose of evaluating feasibility. These methods are not appropriate to determine the *design infiltration rate.*

Site Planning / Design Phase: Where infiltration BMPs are selected, infiltration testing must be conducted to determine the *design infiltration rate* of proposed facilities, except in limited cases where infiltration rate is presumed to be sufficient as identified in Section **VII.1.2**. The required size of the proposed facilities strongly depends on the design infiltration rate; therefore, testing may be required at the preliminary site design phase to facilitate site planning. However, infiltration testing must be conducted as close to the proposed facility as possible, therefore, conducting testing after preliminary site design also has merits. Use of more sophisticated methods at this phase allows better confidence in testing and therefore a lower factor of safety on observed infiltration rates (and therefore smaller facility designs). Factors of safety are discussed in **VII.4**.

Soil characterization and infiltration testing can be considered to fulfill two functions:

1. Determine where infiltration is potentially feasible and must be considered (if other limitations, such as depth to groundwater or contamination, do not restrict infiltration). This role is satisfied through simple infiltration tests, or use of maps and available data.
2. Determine the design infiltration rate for proposed facilities. This function is satisfied through more sophisticated investigation methods, conducted by a qualified professional.

Table VII.1 provides required methods of assessing infiltration rate for each purpose.

Table VII.1: Recommended Infiltration Investigation Methods

Methods for Identifying Areas Potentially Feasible for Infiltration	• **Use of Regional Maps and "Available Data"**[1] **OR** • **Simple Open Pit Infiltration Test** **OR** • Any of the testing methods used to establish design infiltration rate (below)
Methods for Establishing Design Infiltration Rate	• **Open Pit Falling Head Procedure** • **Single Ring Infiltrometer Test** • **Double Ring Infiltrometer Test** • **Well Permeameter Method (USBR Procedure 7300-89)** • **Percolation Test Procedure** (Riverside County Department of Environmental Health) • Other analysis methods at the discretion of the project engineer and approval of the reviewing agency

[1]Available data is defined in Section **VII.2** below and does not require additional investigation.

VII.1.2. Waiver of Infiltration Testing Requirements

The infiltration testing requirements described in this appendix are not applicable for certain combinations of BMP type and general soil condition. In cases where available soils information indicates that the soils are clearly sufficient to support the level of infiltration required for proper function of the BMP and uncertainty in infiltration rate would not significantly influence the performance of the practice, it is not mandatory to conduct infiltration testing. Conditions under which infiltration testing requirements are waived include:

- **Impervious area dispersion** (See **HSC-2: Impervious Area Dispersion**): Testing requirements are waived for this BMP for all soil types. Soil amendments are required to use this practice where site soils are hydrologic soil group C or D.
- **Localized on-lot infiltration** (See **HSC-1: Localized On-Lot Infiltration**): Testing requirements are waived for this BMP for A, B, and C soil types if soil type and general drainage conditions are confirmed with site-specific information. This BMP is not suitable for D soils unless infiltration testing demonstrates that the ponded depth would drain within 24 hours.
- **Porous pavement designed to be self-retaining** (See **INF-6: Permeable Pavement (concrete, asphalt, and pavers)**): Testing requirements for this BMP are waived for A, B, and C soil types if soil type and general drainage conditions are confirmed with site-specific information. This waiver does not apply to porous pavement that accepts run-on from a tributary area larger than 50 percent of its area.
- **Bioinfiltration** (See **INF-4: Bioinfiltration Fact Sheet**). Based on the LID BMP hierarchy, this type of BMP may only be used if infiltration of the full DCV is not feasible; therefore exploratory infiltration rate assessment (Section **VII.2**) is required. However, testing to determine design infiltration rate (Section **VII.3**) is not required. See **Appendix XI** for instructions for sizing the infiltration component of a bioinfiltration BMP to achieve maximum feasible infiltration.

VII.1.3. A Note on "Infiltration Rate" vs. "Percolation Rate"

A common misunderstanding is 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 because they incorporate both downward and horizontal fluxes of water, whereas infiltration only refers to a downward flux of water. When using borehole-type methods, the percolation rate obtained shall be converted to a reasonable estimate of the infiltration rate using the Porchet Method (aka Inverse Borehole Method) (See Example VII.1).

VII.1.4. 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.

VII.1.5. 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) USBR 7300-89, "Procedure for Performing field Permeability Testing by the Well Permeameter Method" (Section VII.3.7 below). Note that this result must be converted to an infiltration rate.
2) The percolation test (Section VII.3.8 below). Note that this result must be converted to an infiltration rate.

VII.1.6. 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.

For shallow fill depths, fill material can be selectively graded to provide reliable infiltration properties. However, in some cases, due to considerable fill depth, the extension of the BMP down to natural soil and selective grading of fill material may prove infeasible. In that case, because of the uncertainty of fill parameters as described above, an infiltration BMP may not be feasible.

VII.2. Methods for Identifying Areas Potentially Feasible for Infiltration

This section describes methods that shall be used, as applicable, to determine whether soils are potentially feasible for infiltration, and where potentially feasible soils exist. Soils would be considered potentially feasible for infiltration if the *measured infiltration rate* obtained from field-testing or obtained by applying professional judgment to available data taken within the Project vicinity is greater than 0.3 inches per hour. *Measured* rates shall account for uncertainty and bias in measurement methods by applying a factor of safety of 2.0 to testing results.

The *measured infiltration rate* calculated for the purpose of infiltration infeasibility screening (TGD Section 2.4) shall be based on a factor of safety of 2.0 applied to the rates obtained from the infiltration test results. No adjustments from this value are permitted. The factor of safety used to compute the *design infiltration rate* shall not be less than 2.0, but may be higher at the discretion of the design engineer and acceptance of the plan reviewer, per the considerations described in Section **VII.4**.

VII.2.1. Use of Regional Maps and "Available Data"

This section describes a method that satisfies the requirements for infiltration screening of small projects as defined by the TGD Infeasibility Screening Criteria (**TGD Section 2.4**). This method uses regionally mapped data coupled with all applicable data available through other site investigations to identify locations not potentially feasible for infiltration as a result of low infiltration rate or high groundwater table.

Via this method, areas of a project identified as having D soils or identified as having depth to first groundwater less than 5 feet are considered infeasible for infiltration if available data confirm these determinations.

Infiltration constraint maps are available in **Appendix XVI** and will be refined as part of the development of Watershed Hydromodification and Infiltration Management Plans. These maps identify constraints, including hydrologic soil group (A,B,C,D), and depth to first groundwater, which should be confirmed through review of available data.

"Available data" is defined as data collected by the project or otherwise available that provides information about infiltration rates and/or groundwater depths. Applicable data is expected to be available as part of nearly all projects subject to New Development and Significant Redevelopment stormwater management requirements in Orange County. Data sources may include:

- Geotechnical investigations
- Due diligence site investigations
- Other CEQA investigations
- Investigations performed on adjacent sites with applicability to the project site

For projects permitted to utilize this method, additional infiltration testing data is not required to be obtained, however, infiltration testing data which is already available from previous studies must be used.

For the purpose of this method, large projects and small projects are defined in Table VII.2. The distinction between large and small projects based the lower spatial variability expected on smaller projects and the lower project value. In these cases, the expense associated with infiltration testing of HSG D soils to attempt to identify localized exceptions to this mapped and supported determination is considered to be an unreasonable economic burden.

Table VII.2: Definition of Project Size Categories

	Residential	Commercial Institutional	Industrial
Small Projects	Less than 10 acres and less than 30 DU	Less than 5 acres and less than 50,000 SF	Less than 2 acre and less than 20,000 SF
Large Projects	Greater than 10 acres or greater than 30 DU	Greater than 5 acres or greater than 50,000 SF	Greater than 2 acre or greater than 20,000 SF

VII.2.2. Simple Open Pit Infiltration Test

The Simple Open Pit Infiltration Test is a site-specific method which can be used to provide a preliminary screening value. This approach cannot be used to find a design infiltration rate. The intent of the Simple Open Pit Infiltration Test is to determine whether or not the local infiltration rate is potentially adequate for LID infiltration BMPs. This approach does not need to be conducted by a licensed professional.

1. The test should be at the proposed facility location or within the immediate vicinity.
2. Excavate a test hole to an elevation 2 feet deeper than the bottom of the infiltration system to account for soil amendment. If the depth of the proposed facility is not known at the time of testing, the excavation should be 6 feet deep. The test hole can be excavated with small excavation equipment or by hand using a shovel, auger, or post hole digger. The hole should be a minimum of 2 feet in diameter and should be sufficient to allow for observation of the water surface level in the bottom of the hole. Remove loose material, as much as possible from the bottom of the hole but avoid compaction of the bottom surface. If a layer hard enough to prevent further excavation is encountered during excavation, or if noticeable moisture/water is encountered in the soil, stop and measure this depth. Proceed with the test at this depth.
3. Fill the hole with water to a height of about 6 inches from the bottom of the hole, and record the exact time. Check the water level at regular intervals (every minute for fast-draining soils to every 10 minutes for slower-draining soils) for a minimum of 1 hour or until all of the water has infiltrated. Record the distance the water has dropped from a fixed reference point such as the top edge of the hole.
4. The infiltration rate is calculated by dividing the change in water elevation time (inches) by the duration of the test (hours).
5. Repeat this process two more times, for a total of three rounds of testing. These tests should be performed as close together as possible to accurately portray the soil's ability to infiltrate at different levels of saturation. The third test provides the best measure of the saturated infiltration rate.

6. For each test pit required, record all three testing results with the date, duration, drop in water height, and conversion into inches per hour.

VII.3. Methods for Establishing Design Infiltration Rate

Allowable methods of establishing design infiltration rate include:

- Open Pit Falling Head Procedure (Section **VII.3.4**)
- Single Ring Infiltrometer Test (Section **VII.3.5**)
- Double Ring Infiltrometer Test (Section **VII.3.6**)
- Well Permeameter Method (USBR Procedure 7300-89) (Section **VII.3.7**)
- Percolation Test Procedure (Riverside County Department of Environmental Health) (Section **VII.3.8**)
- Other analysis methods at the discretion of the project engineer and approval of the reviewing agency

A qualified professional must exercise judgment in the selection of the infiltration test method. Where satisfactory data from adjacent areas is available that demonstrates infiltration testing is not necessary, the infiltration testing requirement may be waived. Waiver of site specific testing is subject to approval by the local approval authority. Recommendation for foregoing infiltration testing must be submitted in a report which includes supporting data and is stamped and signed by the project geotechnical engineer or project geologist.

VII.3.1. Testing Criteria

1. Testing must be conducted or overseen by a qualified professional, either a Professional Engineer (PE) or Registered Geologist (RG) licensed in the State of California.
2. The elevation of the test must correspond to the facility elevation, plus 2 feet to account for soil amendments under the infiltration system. If a confining layer, or soil with a greater percentage of fines, is observed during the subsurface investigation to be within 4 feet of the bottom of the planned infiltration system, the testing should be conducted within that confining layer. The boring log must be continued to a depth adequate to show separation between the bottom of the infiltration facility and the seasonal high groundwater level.
3. Tests must be performed in the immediate vicinity of the proposed facility. Exceptions can be made to the test location provided the qualified professional can support that the strata are consistent from the proposed facility to the test location.
4. Infiltration testing should not be conducted in engineered or undocumented fill.

VII.3.2. Minimum Number of Required Tests

- A total of two infiltration tests for every 10,000 square feet of lot area available for new or redevelopment (minimum 2 tests per priority project).

- An additional test for every 10,000 square feet of lot area available for new or redevelopment.
- At least one test for any potential street facility.
- One test for every 100 lineal feet of infiltration facility.
- No more than five tests are required per development (at the discretion of the qualified professional assessing the site, as well as the reviewing agency).

Where multiple types of facilities are used, it is likely that multiple tests will be necessary, since different facility types may infiltrate at different depths and an infiltration test can test only a single soil stratum. It is highly recommended to conduct an infiltration test at each stratum used. Additional testing may be required at the discretion of the local approval authority.

VII.3.3. 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:

1. Over time, the surface of infiltration facilities can become plugged as sedimentary particles accumulate at the infiltration surface.
2. Post-grading compaction of the site can destroy soil structure and seriously impact the facility's performance.
3. Testing procedures in general are subject to errors which can skew the results.

The method for determination of the factor of safety described in Section VII.4 includes, among other factors, a consideration of the testing methods used to measure infiltration rate. The open pit falling head test (see Section VII.3.4) is considered the most reliable infiltration testing method if constructed to the recommended dimensions.

VII.3.4. Open Pit Falling Head Procedure

The open pit falling head procedure is performed in an open excavation and therefore is a test of the combination of vertical and lateral infiltration. The tester and excavator should conduct all testing in accordance with OSHA regulations regarding open pit excavations.

1. Excavate a hole with bottom dimensions of at least 2 feet by 4 feet into the native soil to the elevation 2 feet below the proposed facility bottom to account for amendment of soils under infiltration areas. If a smooth excavation bucket is used, scratch the sides and bottom of the hole with a sharp pointed instrument, and remove the loose material from the bottom of the test hole. The bottom of the hole should not be compacted and should be as level as possible.
2. Fill the hole with clean water a minimum of 1 foot above the soil to be tested, and maintain this depth of water for at least 4 hours (or overnight if clay soils are present) to

presoak the native material. In sandy soils with little or no clay or silt, soaking is not necessary. If after filling the hole twice with 12 inches of water, the water seeps completely away in less than 10 minutes, the test can proceed immediately.

3. Determine how the water level will be accurately measured. The measurements should be made with reference to a fixed point. A lath placed in the test pit prior to filling or a sturdy beam across the top of the pit are convenient reference points.
4. After the pre-saturation period, refill the hole with water to 12 inches above the soil and record the time. For deep holes, it may be necessary to use remote sensing equipment to accurately measure changes in water level. Alternative water head heights may be used for testing provided the presaturation height is adjusted accordingly and the water head height used in infiltration testing is 50 percent or less than the water head height in the proposed stormwater system during the design storm event. Measure the water level to the nearest 0.01 foot ($\frac{1}{8}$ inch) at 10-minute intervals for a total period of 1 hour (or 20-minute intervals for 2 hours in slower soils) or until all of the water has drained. In faster draining soils (sands and gravels), it may be necessary to shorten the measurement interval in order to obtain a well-defined infiltration rate curve. Constant head tests may be substituted for falling head tests at the discretion of the professional overseeing the infiltration testing.
5. Repeat the test. Successive trials should be run until the percent change in measured infiltration rate between two successive trials is minimal (<10 percent). The trial should be discounted if the infiltration rate between successive trials increases. At least three trials must be conducted. After each trial, the water level is readjusted to the 12 inch level. Record results.
6. The average infiltration rate over the last trial should be used to calculate the unadjusted (pre-factor of safety) infiltration rate. The final rate must be reported in inches per hour.
7. Upon completion of the testing, the excavation must be backfilled.
8. For very rapidly draining soils, it may not be possible to maintain a water head above the bottom of the test pit. If the infiltration rate meets or exceeds the flow of water into the test pit, conduct the test in the following manner:
    a) Approximate the area over which the water is infiltrating.
    b) Using a water meter, bucket, or other device, measure the rate of water discharging into the test pit.
    c) Calculate the infiltration rate by dividing the rate of discharge (cubic inches per hour) by the area over which it is infiltrating (square inches) and correcting to units of inches per hour.

VII.3.5. Single Ring Infiltrometer Test

Single ring infiltrometer tests using a large ring in diameter (40 inches or larger is optimal) have been shown to closely match full-scale facility performance (**Figure VII.1** to **Figure VII.3**). The cylindrical ring is driven approximately 12 inches into the soil. Water is ponded within the ring above the soil surface. The upper surface of the ring is often covered to prevent evaporation. Using the constant head method, the volumetric rate of water added to the ring sufficient to maintain a constant head within the ring is measured. The test is complete and the tested infiltration rate, I_t, is determined after the flow rate has stabilized (ASTM D5126).

To help maintain a constant head, a variety of devices may be used. A hook gage, 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:

$$I_t = V/(A^*t)$$

where:

I_t = tested infiltration rate, in/hr
V = volume of liquid used during time interval to maintain constant head in the ring, in^3
A = internal area of ring, in^2
t = time interval, hr.

Figure VII.1. Photo of Single Ring Infiltrometer

Figure VII.2. Single Ring Infiltrometer Construction

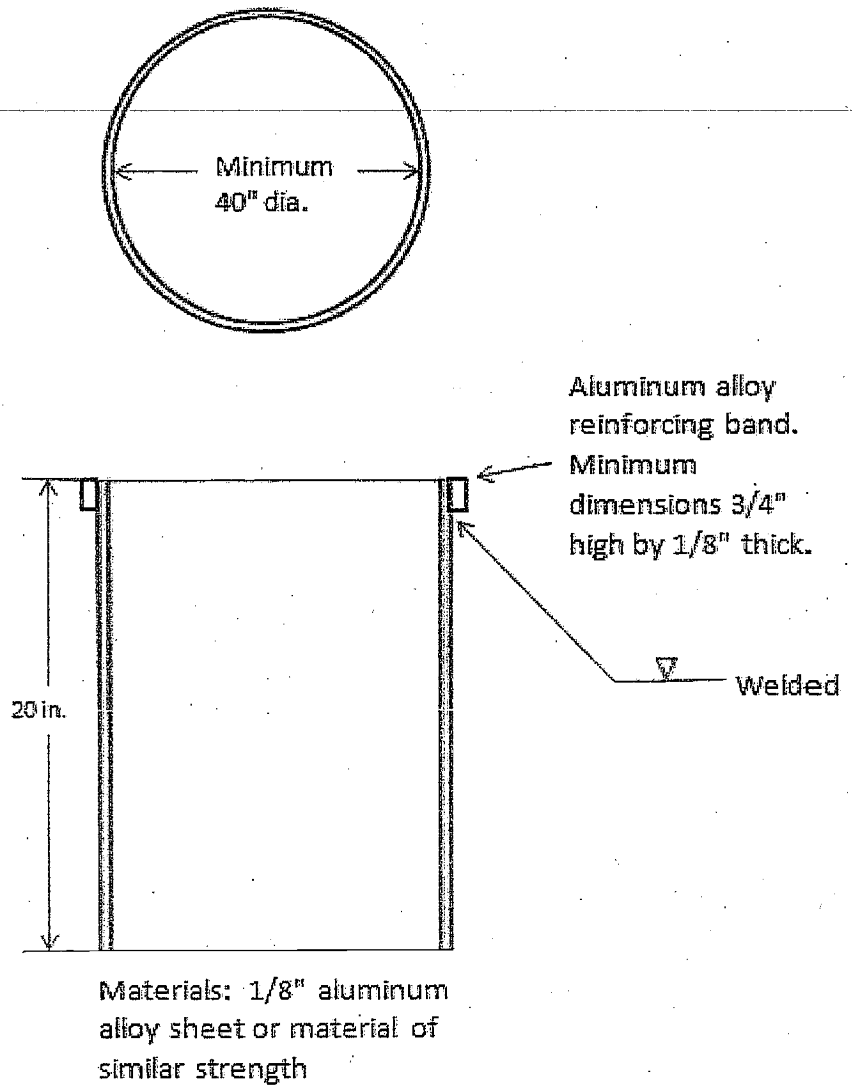


Figure VII.3. Single Ring Infiltrometer Setup with Mariotte Tube

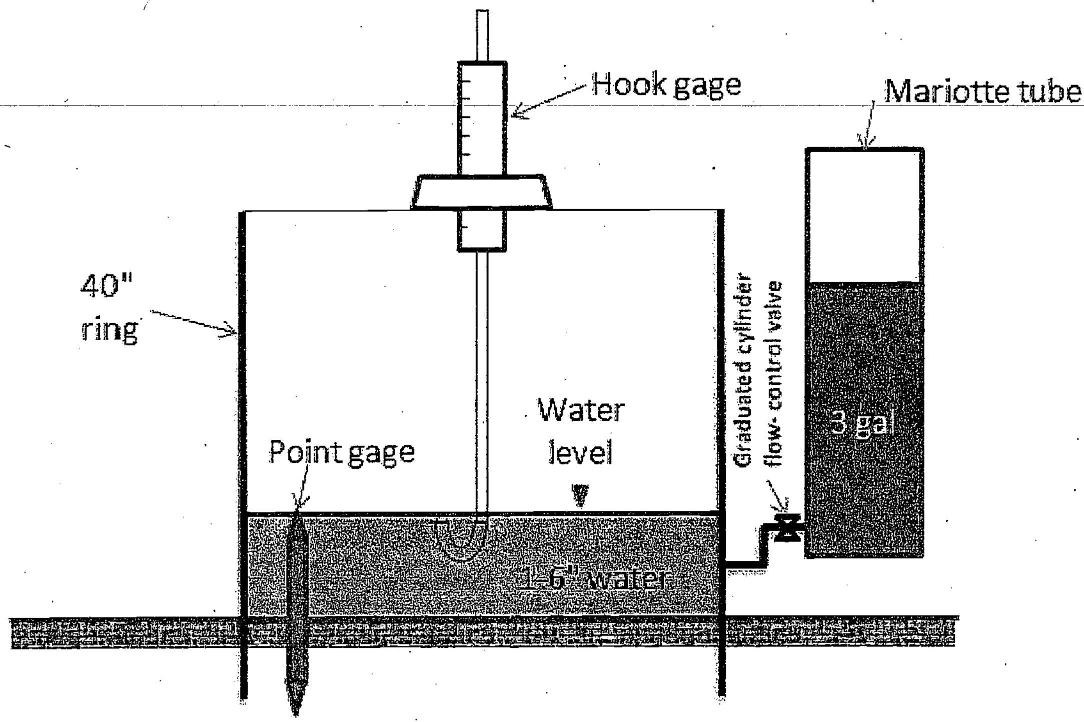


Figure VII.4. Sample Test Data Form for Single Ring Infiltrometer Test

SINGLE RING INFILTROMETER TEST DATA

Project Name and Test Location:	Constants-	Ring Data: Ring Area, A_r (in²)	Ring Data: Depth of Liquid (in)	Liquid Containers: Reservoir Container Volume, V_r, (in³/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_r* (in³)	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, $Q_r = \Delta H \times V_r$ **Infiltration Rate, $I = (Q_r/A_r)/$

VII.3.6. Double Ring Infiltrometer Test

The double ring infiltrometer test (ASTM D3385) is a well-recognized and documented technique for directly measuring the soil infiltration rate of a site (see **Figure VII.5** to **Figure VII.12**). 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. The double-ring infiltrometer test should be performed at an elevation 2 feet below the proposed elevation of the infiltration surface to account for the use of soil amendments below the infiltration system.

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 gage, 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:

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

where:

> I_t = tested infiltration rate, in/hr
> V = volume of liquid used during time interval to maintain constant head in the inner ring, in^3
> A = area of inner ring, in^2
> t = time interval, hr.

Figure VII.5. Photo of Simple Double Ring Infiltrometer

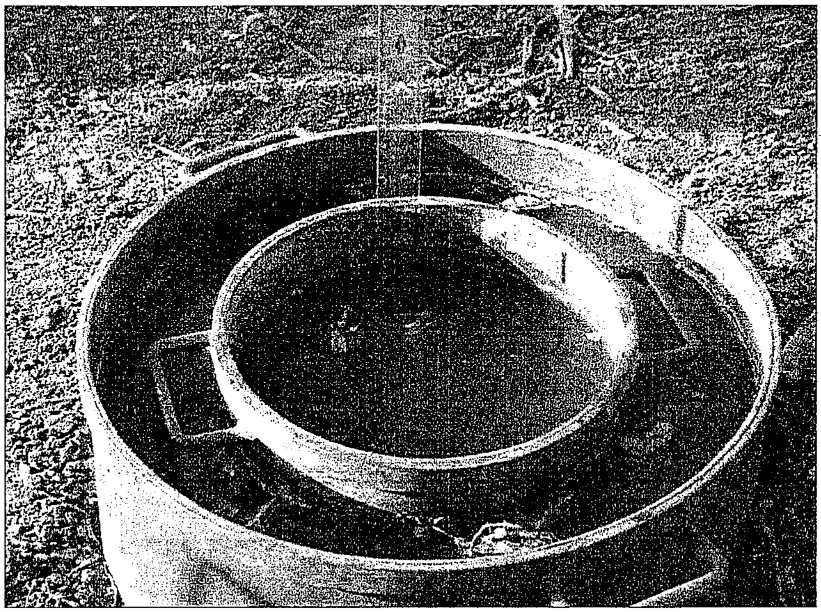
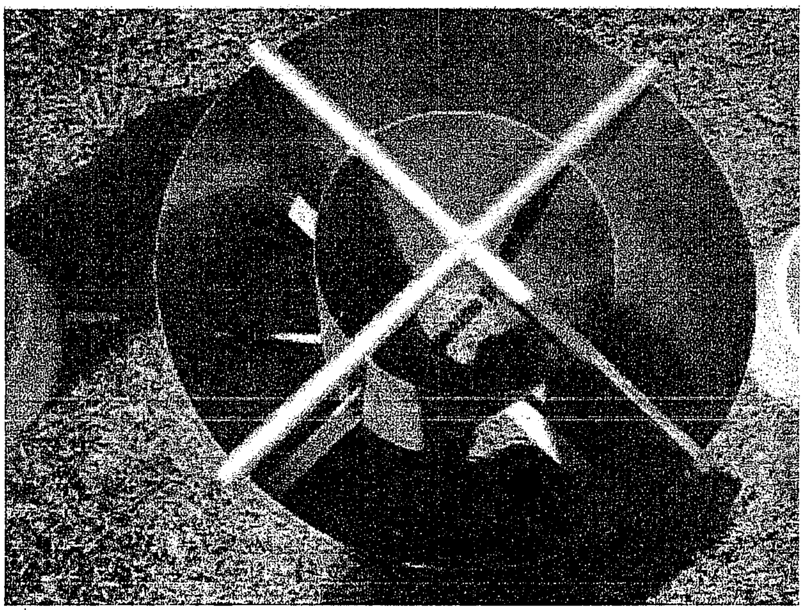
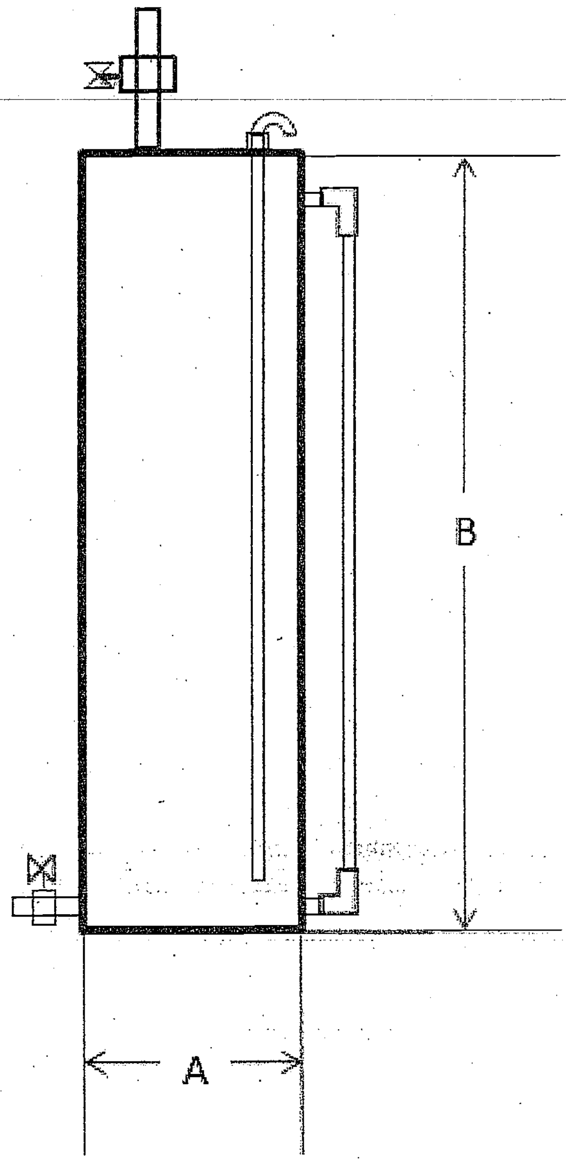


Figure VII.6. Photo of Pre-fabricated Double Ring Infiltrometer



(Photo courtesy of Turf-Tec International)

Figure VII.7. Mariotte Tube



Mariotte Tube
Useful Capacity

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

Figure VII.8. Double Ring Infiltrometer Construction

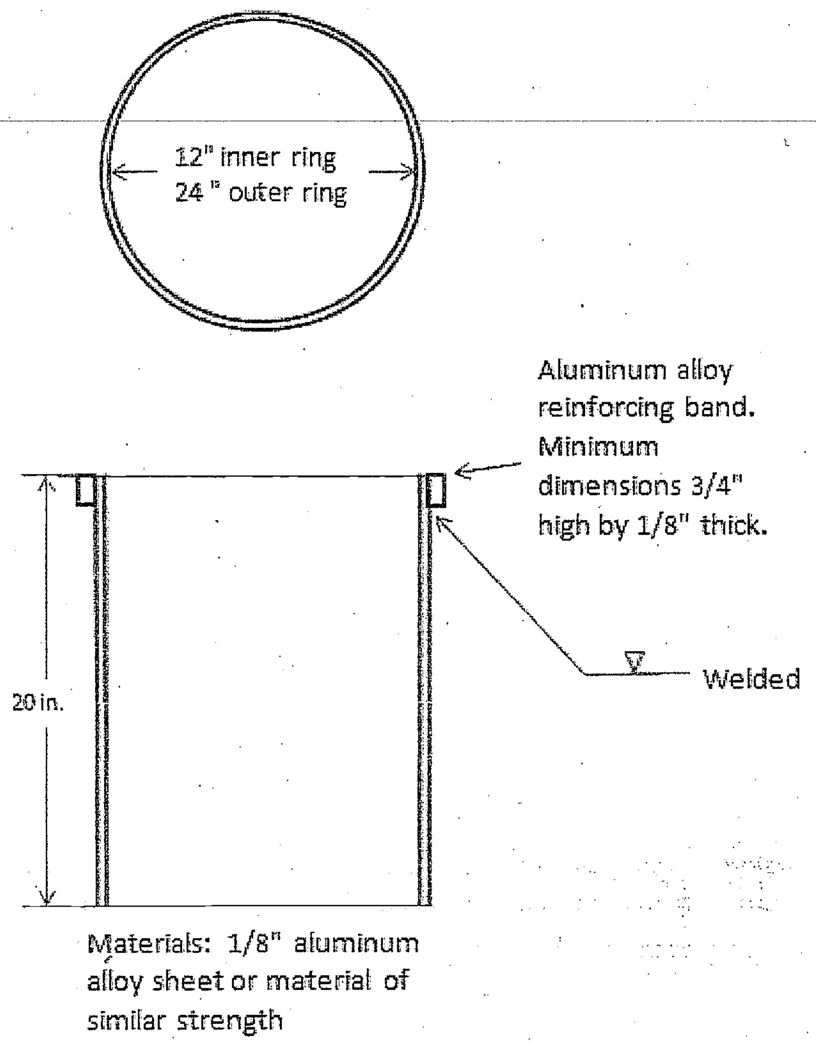


Figure VII.9. Double Ring Setup with Mariotte Tubes

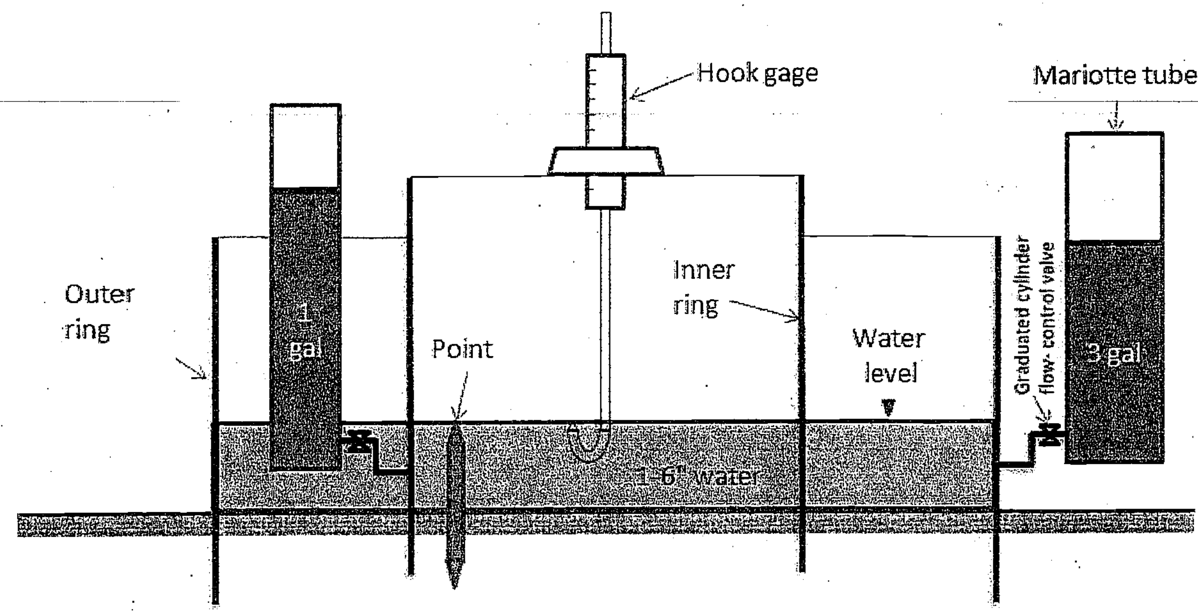
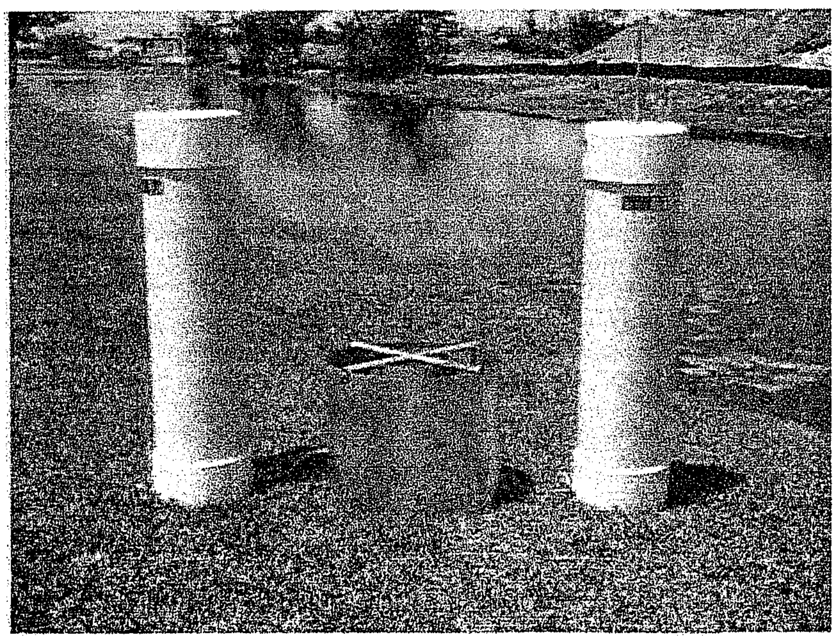


Figure VII.10. Double Ring Infiltrometer Set-up with Mariotte Tubes



(Photo courtesy of Turf-Tec International)

Figure VII.11. Double Ring Infiltrometer Set-up for Test at Basin Surface Elevation

(Photo courtesy of Turf-Tec International)

Figure VII.12. 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)
			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										
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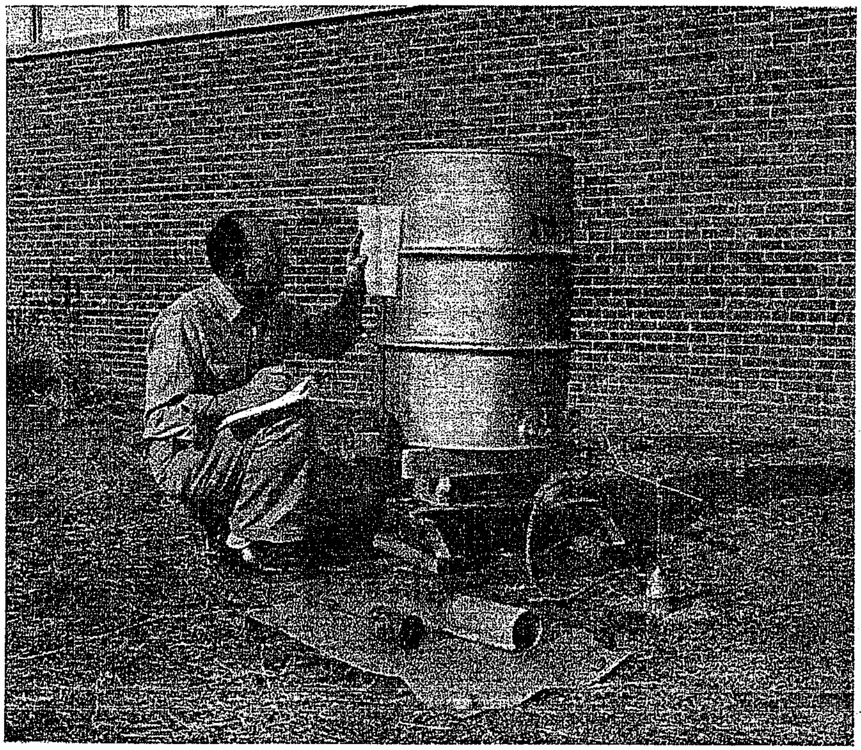
10 - Start										
End										
11 - Start										
End										
12 - Start										
End										
13 - Start										
End										
14 - Start										
End										
15 - Start										
End										

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

VII.3.7. Well Permeameter Method (USBR Procedure 7300-89)

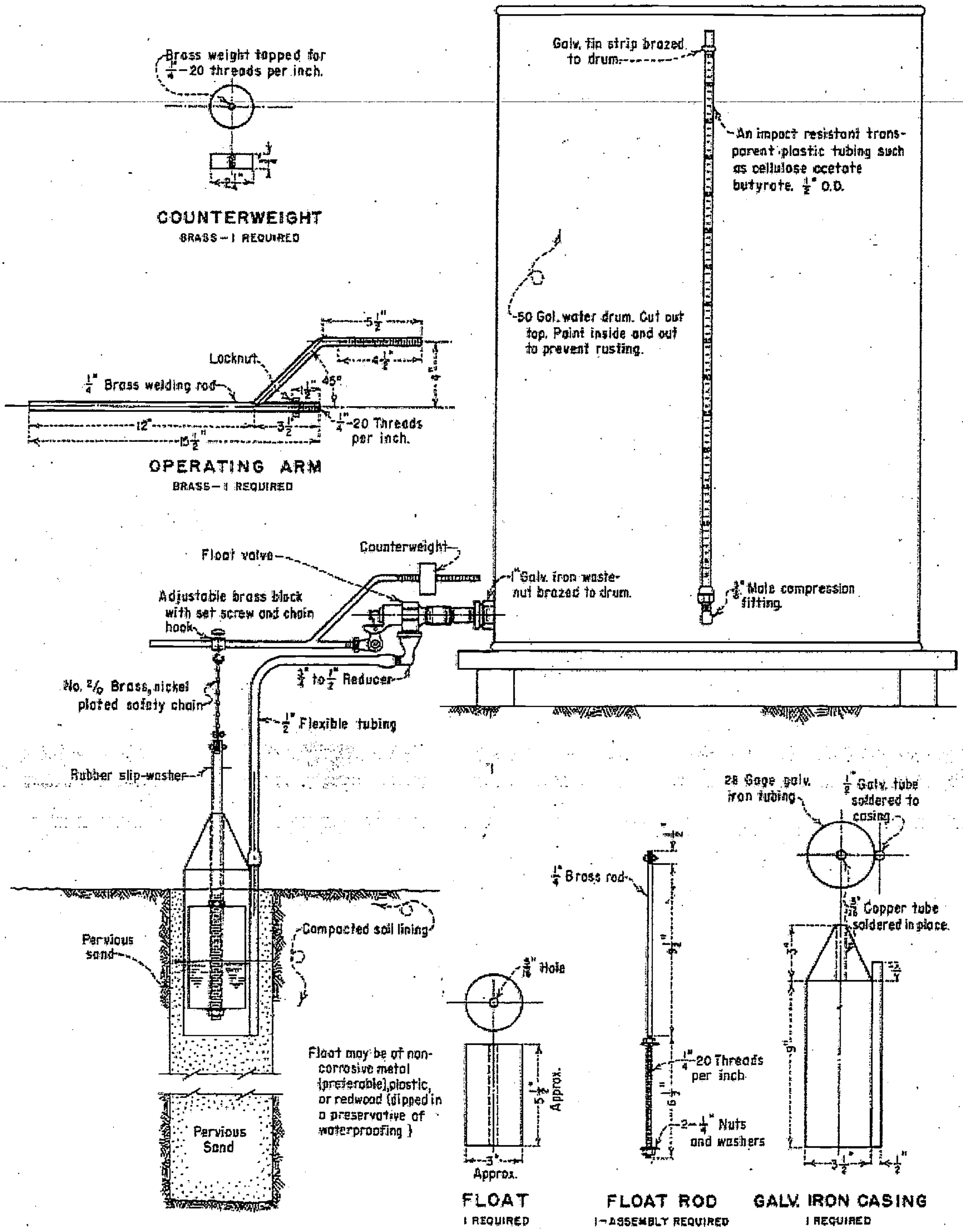
Similar to a constant-head version of the percolation test used for seepage pit design is the Well Permeameter Method of the United States Bureau of Reclamation (see **Figure VII.13** and **Figure VII.14**). [12]USBR 7300-89 is an in-hole hydraulic conductivity test performed by drilling test wells with 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 not an infiltration rate, but this procedure uses special equation(s) to establish an infiltration rate from the data produced. See USBR procedure 7300-89 for more details.

Figure VII.13. Typical Well Permeameter Test Installation



[12] 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.

Figure VII.14. Well Permeameter Test Equipment



VII.3.8. Percolation Test Procedure

The percolation test procedure below (per Riverside County Department of Environmental Health) should only be performed by those individuals trained and educated to perform, understand and evaluate the field conditions and 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 Hydrogeologist.

The procedure for this test varies, depending on the depth of the hole to be used. Procedures for both scenarios (less than 10 feet or 10 - 40 feet deep) and diagrams (**Figure VII.15** to **Figure VII.17**) are included below. 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.

VII.3.8.1. Shallow Percolation Test (less than 10 feet)

Test Preparation

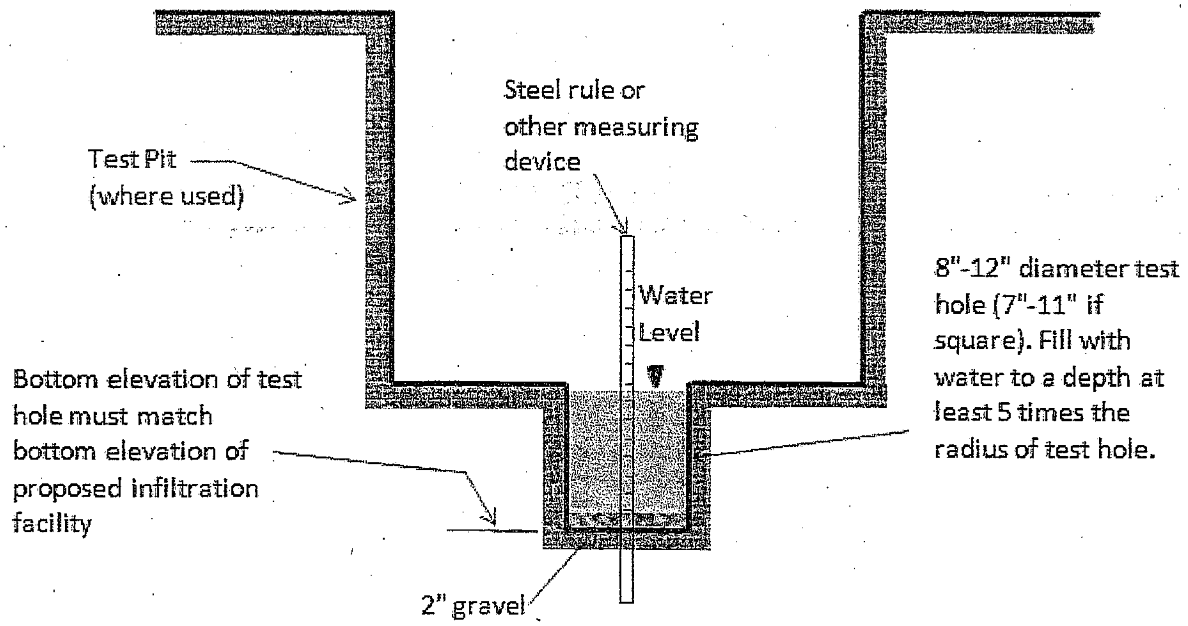
1) The test hole opening shall be between 8 and 12 inches in diameter or between 7 and 11 inches on each side if square.
2) The bottom elevation of the test hole shall correspond to the bottom elevation of the proposed basin (infiltration surface). Keep in mind that this procedure will require the test 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 **sandy soils**, 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 VII.15. Test Pit for Shallow Percolation Test



VII.3.8.2. Deep Percolation Test (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.

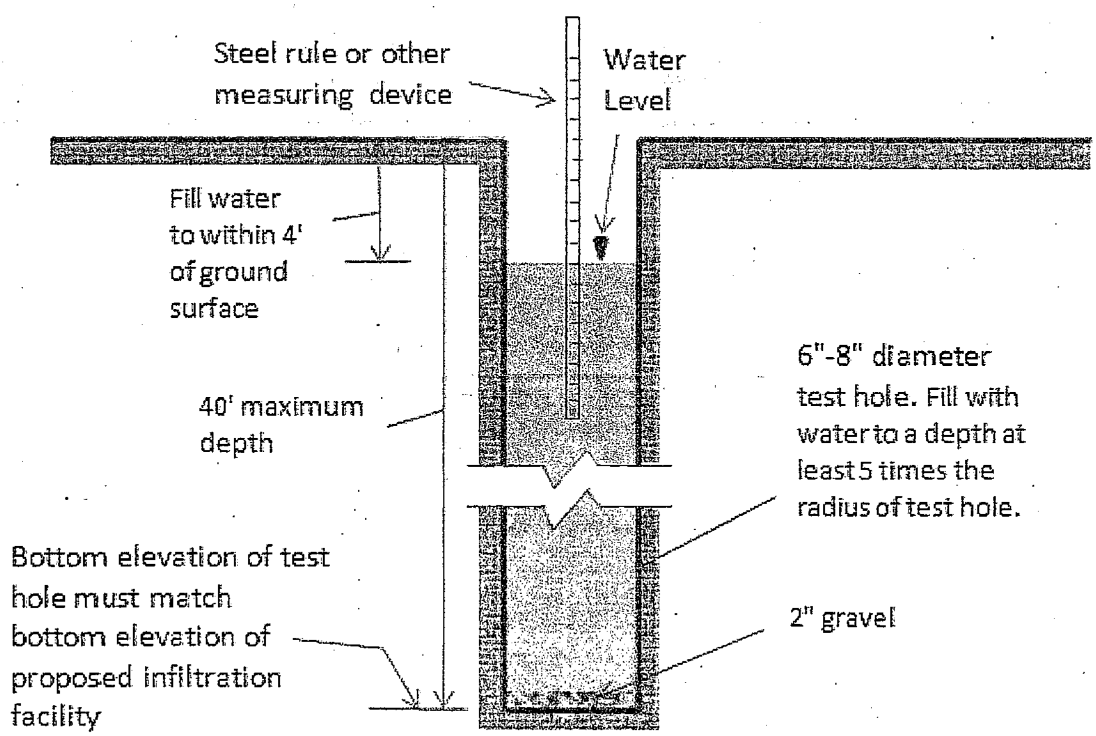
2) The bottom elevation of the test hole shall correspond to the bottom elevation of the proposed basin (infiltration surface). Keep in mind that this procedure will require the test hole to be filled with water to a depth of at least 5 times the hole's radius.

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 "continuous pre-soak procedure" is not accepted. When sandy soils (as described below) are present, the test shall be run immediately.

Figure VII.16. 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 sandy **soils**, 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**, 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.

Figure VII.17. Photo of Percolation Test Pit.

(Use of perforated PVC pipe is a variation.)

Figure VII.18. Sample Test Data Form for Percolation Test

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:							

Example VII.1: Percolation Rate Conversion Example

(Porchet Method, aka Inverse Borehole Method):

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. The data collected at the final interval is as follows:

Time interval, $\Delta t = 10$ minutes
Initial Depth to Water, $D_0 = 12.25$ inches
Final Depth to Water, $D_f = 13.75$ inches
Total Depth of Test Hole, $D_T = 60$ inches
[13]Test Hole Radius, r = 4 inches

The conversion equation is used:

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

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

$H_o = D_T - D_0 = 60 - 12.25 = 47.75$ inches

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

$H_f = D_T - D_0 = 60 - 13.75 = 46.25$ inches

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

$\Delta H = \Delta D = H_o - H_f = 47.75 - 46.25 = 1.5$ inches

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

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

"I_t" is the tested infiltration rate.

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

[13] 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., $r = (A/\pi)^{0.5}$).

VII.4. Considerations for Infiltration Rate Factor of Safety

Given the known potential for infiltration BMPs to fail over time, an appropriate factor of safety applied to infiltration testing results must be mandatory. The infiltration rate will decline between maintenance cycles as the BMP surface becomes occluded and particulates accumulate in the infiltrative layer. Monitoring of actual facility performance has shown that the full-scale infiltration rate is far lower than the rate measured by small-scale testing. It is important that adequate conservatism is incorporated in the selection of design infiltration rates. The design infiltration rate discussed here is the infiltration rate of the underlying soil, below the elevation to which soil amendments would not be provided.

The factor of safety that should be applied to measured infiltration rates is a function of:

- Suitability of underlying soils for infiltration
- The infiltration system design.

These factors are discussed in the following sections.

The *measured infiltration rate* calculated for the purpose of infiltration infeasibility screening (**TGD Section 2.4**) shall be based on a factor of safety of 2.0 applied to the rates obtained from the infiltration test results. No adjustments from this value are permitted. The factor of safety used to compute the *design infiltration rate* shall not be less than 2.0, but may be higher at the discretion of the design engineer and acceptance of the plan reviewer, per the considerations described in the following sections.

It is recognized that there are competing objectives in the selection of a factor of safety. There is an initial economic incentive to select a lower factor of safety to yield smaller BMP designs. A low factor of safety also allows a broader range of systems to be considered "feasible" in marginal conditions. However, there are both economic and environmental incentives for the use of an appropriate factor of safety to prevent premature failure and substandard performance. The use of an artificially low factor of safety to demonstrate feasibility in the design process is shortsighted in that it does not consider the long term feasibility of the system.

The best way to balance these competing factors is through a commitment to thorough site investigation, use of effective pretreatment controls, good construction practices, the commitment to restore the infiltration rates of soils that are damaged by prior uses or construction practices, and the commitment to effective maintenance practices. However, these commitments do not mitigate the need to apply a factor of safety to account for uncertainty and long term deterioration that cannot be technically mitigated. Therefore, a factor of safety of no less than 2.0 shall be used to compute the design infiltration rate.

VII.4.1. Site Suitability Considerations

Suitability assessment related considerations include (**Table VII.3**):

- Soil assessment methods – the site assessment extent (e.g., number of borings, test pits, etc.) and the measurement method used to estimate the short-term infiltration rate.
- Predominant soil texture/percent fines – soil texture and the percent of fines can greatly influence the potential for clogging.
- Site soil variability – site with spatially heterogeneous soils (vertically or horizontally) as determined from site investigations are more difficult to estimate average properties for resulting in a higher level of uncertainty associated with initial estimates.
- Depth to seasonal high groundwater/impervious layer – groundwater mounding may become an issue during excessively wet conditions where shallow aquifers or shallow clay lenses are present.

Table VII.3: Suitability Assessment Related Considerations for Infiltration Facility Safety Factors

Consideration	High Concern	Medium Concern	Low Concern
Assessment methods (see explanation below)	Use of soil survey maps or simple texture analysis to estimate short-term infiltration rates	Direct measurement of ≥ 20 percent of infiltration area with localized infiltration measurement methods (e.g., infiltrometer)	Direct measurement of ≥ 50 percent of infiltration area with localized infiltration measurement methods or Use of extensive test pit infiltration measurement methods
Texture Class	Silty and clayey soils with significant fines	Loamy soils	Granular to slightly loamy soils
Site soil variability	Highly variable soils indicated from site assessment or limited soil borings collected during site assessment	Soil borings/test pits indicate moderately homogeneous soils	Multiple soil borings/test pits indicate relatively homogeneous soils
Depth to groundwater/ impervious layer	<5 ft below facility bottom	5-10 ft below facility bottom	>10 below facility bottom

Localized infiltration testing refers to methods such as the double ring infiltrometer test (ASTM D3385-88) which measure infiltration rates over an area less than 10 sq-ft, may include lateral flow, and do not attempt to account for heterogeneity of soil. The amount of area each test represents should be estimated depending on the observed heterogeneity of the soil.

Extensive infiltration testing refers to methods that include excavating a significant portion of the proposed infiltration area, filling the excavation with water, and monitoring drawdown. The excavation should be to the depth of the proposed infiltration surface and ideally be at least 50 to 100 square feet.

In all cases, testing should be conducted in the area of the proposed BMP where, based on review of available geotechnical data, soils appear least likely to support infiltration.

VII.4.2. Design Related Considerations

Design related considerations include (**Table VII.4**):

- Size of area tributary to facility – all things being equal, risk factors related to infiltration facilities increase with an increase in the tributary area served. Therefore facilities serving larger tributary areas should use more restrictive adjustment factors.
- Level of pretreatment/expected influent sediment loads – credit should be given for good pretreatment by allowing less restrictive factors to account for the reduced probability of clogging from high sediment loading. Also, facilities designed to capture runoff from relatively clean surfaces such as rooftops are likely to see low sediment loads and therefore should be allowed to apply less restrictive safety factors.
- Redundancy – facilities that consist of multiple subsystems operating in parallel such that parts of the system remains functional when other parts fail and/or bypass should be rewarded for the built-in redundancy with less restrictive correction and safety factors. For example, if bypass flows would be at least partially treated in another BMP, the risk of discharging untreated runoff in the event of clogging the primary facility is reduced. A bioretention facility that overflows to a landscaped area is another example.
- Compaction during construction – proper construction oversight is needed during construction to ensure that the bottoms of infiltration facility are not overly compacted. Facilities that do not commit to proper construction practices and oversight should have to use more restrictive correction and safety factors.

Table VII.4: Design Related Considerations for Infiltration Facility Safety Factors

Consideration	High Concern	Medium Concern	Low Concern
Tributary area size	Greater than 10 acres.	Greater than 2 acres but less than 10 acres.	2 acres or less.
Level of pretreatment/ expected influent sediment loads	Pretreatment from gross solids removal devices only, such as hydrodynamic separators, racks and screens AND tributary area includes landscaped areas, steep slopes, high traffic areas, or any other areas expected to produce high sediment, trash, or debris loads.	Good pretreatment with BMPs that mitigate coarse sediments such as vegetated swales AND influent sediment loads from the tributary area are expected to be relatively low (e.g., low traffic, mild slopes, disconnected impervious areas, etc.).	Excellent pretreatment with BMPs that mitigate fine sediments such as bioretention or media filtration OR sedimentation or facility only treats runoff from relatively clean surfaces, such as rooftops.
Redundancy of treatment	No redundancy in BMP treatment train.	Medium redundancy, other BMPs available in treatment train to maintain at least 50% of function of facility in event of failure.	High redundancy, multiple components capable of operating independently and in parallel, maintaining at least 90% of facility functionality in event of failure.
Compaction during construction	Construction of facility on a compacted site or elevated probability of unintended/ indirect compaction.	Medium probability of unintended/ indirect compaction.	Heavy equipment actively prohibited from infiltration areas during construction and low probability of unintended/ indirect compaction.

VII.4.3. Determining Factor of Safety

A factor of safety is shall be used. To assist in selecting the appropriate design infiltration rate, the measured short term infiltration rate should be adjusted using a weighted average of several safety factors using the worksheet shown in Worksheet H below. The design infiltration rate would be determined as follows:

- For each consideration shown in **Table VII.3** and **Table VII.4** above, determine whether the consideration is a high, medium, or low concern.

For all high concerns, assign a factor value of 3, for medium concerns, assign a factor value of 2, and for low concerns assign a factor value of 1.
Multiply each of the factors by the corresponding weight to get a product.
Sum the products within each factor category to obtain a safety factor for each.
Multiply the two safety factors together to get the final combined safety factor. If the combined safety factor is less than 2, then 2 shall be used as the safety factor.
Divide the measured short term infiltration rate by the combined safety factor to obtain the adjusted design infiltration rate for use in sizing the infiltration facility.

The design infiltration rate shall be used to size BMPs and to evaluate their expected long term performance. This rate shall not be less than 2, but may be higher at the discretion of the design engineer.

Worksheet H: Factor of Safety and Design Infiltration Rate and Worksheet

Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) $p = w \times v$
A	Suitability Assessment	Soil assessment methods	0.25		
		Predominant soil texture	0.25		
		Site soil variability	0.25		
		Depth to groundwater / impervious layer	0.25		
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			
B	Design	Tributary area size	0.25		
		Level of pretreatment/ expected sediment loads	0.25		
		Redundancy	0.25		
		Compaction during construction	0.25		
		Design Safety Factor, $S_B = \Sigma p$			
Combined Safety Factor, $S_{TOT} = S_A \times S_B$					
Measured Infiltration Rate, inch/hr, K_M (corrected for test-specific bias)					
Design Infiltration Rate, in/hr, $K_{DESIGN} = S_{TOT} \times K_M$					
Supporting Data					
Briefly describe infiltration test and provide reference to test forms:					

Note: The minimum combined adjustment factor shall not be less than 2.0 and the maximum combined adjustment factor shall not exceed 9.0.

VII.5. References

ASTM D 3385-94, 2003. "Standard Test Method for Infiltration Rate of Soils Field Using Double-Ring Infiltrometer." American Society for Testing Materials, Conshohocken, PA. 10 Jun, 2003.

Caltrans, 2003. "Infiltration Basin Site Selection". Study Volume I. California Department of Transportation. Report No. CTSW-RT-03-025.

City of Portland, 2010. *Appendix F.2: Infiltration Testing*. Portland Stormwater Management Manual, Revised February 1, 2010.

United States Department of the Interior, Bureau of Reclamation (USBR), 1990a, "Procedure for Performing Field Permeability Testing by the Well Permeameter Method (USBR 7300-89)," in Earth Manual, Part 2, A Water Resources Technical Publication, 3rd ed., Bureau of Reclamation, Denver, Colo.

APPENDIX VIII. GROUNDWATER-RELATED INFILTRATION FEASIBILITY CRITERIA

Infiltration BMPs shall not be used where they would adversely affect groundwater quality or where depth to groundwater would limit infiltration. The purpose of this section is to provide guidelines for allowable use of infiltration BMPs to protect groundwater quality and ensure physical feasibility relative to groundwater and groundwater-related geotechnical considerations. This section considers:

- Depth to groundwater and mounding potential,
- Presence of groundwater plumes,
- Wellhead protection and septic systems,
- Contamination risks from land use activities in the area tributary to the BMP,
- Consultation with applicable groundwater agencies, and
- Technical requirements for conducting site specific studies,

VIII.1. Intended Use

The criteria contained in this section are intended to be used as part of the overall feasibility screening process. If other feasibility criteria (e.g., low soil infiltration rate) render infiltration infeasible, it is not necessary to also consider the criteria contained in this section. However, before infiltration BMPs are approved for use on a project, these groundwater quality-related criteria must be evaluated.

VIII.2. Depth to Groundwater and Mounding Potential

Minimum separation between the infiltrating surface (bottom of infiltration facility) and seasonally high mounded groundwater shall be observed in the design of infiltration BMPs, depending on BMP type.

- If the depth to unmounded seasonally high groundwater is greater than 15 feet, the depth to groundwater does not constrain infiltration
- If separation to unmounded seasonally high groundwater is greater than 10-feet and the infiltration area is less than 2,000 sq-ft, the depth to groundwater does not constrain infiltration.
- The separation between the infiltrating surface and the seasonally high mounded groundwater table shall not be less than 5 feet for all BMP types. BMPs for which 5-foot minimum separation applies include:

- Rain gardens and dispersion trenches (small, residential applications)
- Bioretention and planters
- Permeable Pavement
- Similar BMPs infiltrating over an extensive surface area and providing robust pretreatment or embedded treatment processes.

- Separation to mounded seasonally high groundwater shall be at least 10 feet for infiltration devices that inject water below the subsurface and surface infiltration BMPs with tributary area and land use activities that are considered to pose a more significant risk to groundwater quality. BMPs for which the 10-foot separation applies include:
  - Dry wells
  - Subsurface infiltration galleries or vaults
  - Surface Infiltration Basins
  - Infiltration Trenches
  - Other functionally similar devices or BMPs.

VIII.2.1. Approved Methods for Determining the Depth to Seasonally High Groundwater

The seasonally high groundwater table is defined as the depth to the highest level of the saturated groundwater zone. It is quantified as the average of measured annual minima (i.e., the shallowest recorded measurements in each water year, defined as October 1 through September 30 are averaged) for all years on record.

The depth to seasonally high groundwater is ideally determined from long-term groundwater level data. If groundwater level data are not available or are inadequate, the seasonal high groundwater depth can be estimated by redoximorphic analytical methods combined with temporary groundwater monitoring for November 1 through April 1 at the proposed Project site. In this approach, a professional geologist assesses soil-mottling characteristics of soil cores to determine the depth at which soil features display reductive conditions which indicate the seasonal height of groundwater.

VIII.2.2. Methods for Evaluation of Groundwater Mounding Potential

Stormwater infiltration and recharge to the underlying groundwater table will in most cases create a groundwater mound beneath the infiltration facility. The height and shape of the mound depends on the infiltration system design, the recharge rate, and the hydrogeologic conditions at the site, especially the horizontal hydraulic conductivity and the saturated thickness. Groundwater mounding beneath infiltration facilities also depends on the precipitation patterns, which affects the applied recharge rates and underlying soil moisture conditions. Maximum mounding potential is likely to occur in response to cumulative precipitation over relatively short periods, for example, a series of intense winter storms over a one to two week period.

Methods for quantifying groundwater mounding potential range from detailed modeling studies to simple conservative estimation techniques. The methods employed will be selected by the project proponent to the acceptance of the reviewing agency.

Mounding Evaluation with Modeling Studies: A rigorous evaluation of mounding potential requires detailed site characterization and detailed modeling that accounts for the transient nature of stormwater infiltration and the site-specific hydrogeological conditions. For example, Carlton (2010)[14] used MODFLOW, an industry standard groundwater flow model, to evaluate groundwater mounding potential from infiltration facilities in hypothetical 1-acre and 10-acre developments. Modeling studies to evaluate groundwater mounding potential are applicable for design studies of large regional facilities. Detailed modeling analyses are typically not feasible for evaluation of on-site facilities in small development projects or dispersed small-scale facilities in larger projects.

Mounding Estimates Based on Simplified Groundwater Equations: Estimates of maximum mounding potential can be developed from analytical solutions to groundwater equations, called the Hantush equations. These equations incorporate a number of simplifying assumptions about the hydrogeology of the site including assumptions of uniform horizontal hydraulic conductivity and vertical infiltration rates. Solution of the Hantush equations can be accomplished with a simple Excel spreadsheet tool developed by the USGS (Carlton, 2010) available at online at **http://pubs.usgs.gov/sir/2010/5102/**.

This tool is simple to use but requires inputs about the saturated zone hydraulic conductivity, the thickness of the saturated zone, and estimates of the specific yield, which is related to the effective porosity. The tool also requires inputs about the infiltration conditions, including the dimensions of the infiltration facility, the uniform infiltration rate and the period application that will result in the maximum mounding height. Use of the USGS groundwater mounding tool is applicable and recommended for planning or design level analysis where there is the sufficient information of the surface conditions of the site and use of detailed modeling is not warranted.

Where information is not available, the following assumptions are recommended for using this tool to evaluating the potential for mounding under small-scale localized BMPs. Site-specific data and professional judgment should always be used in conducting groundwater mounding analyses.

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[14] Carleton, G.B., 2010, Simulation of groundwater mounding beneath hypothetical stormwater infiltration basins: U.S. Geological Survey Scientific Investigations Report 2010–5102, 64 p. http://pubs.usgs.gov/sir/2010/5102/

- Recharge rate should be set to the design infiltration rate of the stormwater BMP, assuming that the BMP operates at its design infiltration rate throughout the critical period for groundwater mounding.
- The horizontal hydraulic conductivity should be set to 10 times the measured infiltration rate of the soil to account for typical anisotropy of natural soils (ratio of horizontal to vertical hydraulic conductivity). Note the measured infiltration rate will generally be greater than or equal to 2 times the design infiltration rate.
- The period of simulation should be set to 10 days. Applying the design infiltration rate continuously over 10 days generally results in 3-5 times the DCV infiltrated over this period considering typical BMP drawdown times.
- The specific yield should be set to 0.2.
- The saturated zone thickness should be set to 20 feet.

An example using the USGS tool is included in Example VIII.1 below.

Example VIII.1: Application of USGS Groundwater Mounding Tool Using a Hypothetical Range of Infiltration Scenarios

Given:

- Measured soil infiltration rate: 0.2 to 4 inches per hour
- Design infiltration rate: 0.1 to 2 inches per hour (Factor of Safety = 2.0)
- Horizontal Hydraulic Conductivity: 2 to 40 inches per hour (Anisotropy: 10:1 (H:V) applied to measured infiltration rate)
- Facility footprint: 500 to 4,000 sq-ft
- *System aspect ratio*: 1:1 (square) and 5:1
- Period of simulation: 10 days (total infiltrated depth =24 to 480 inches)
- Saturated zone thickness: 20 feet
- Specific yield: 0.2

Required:

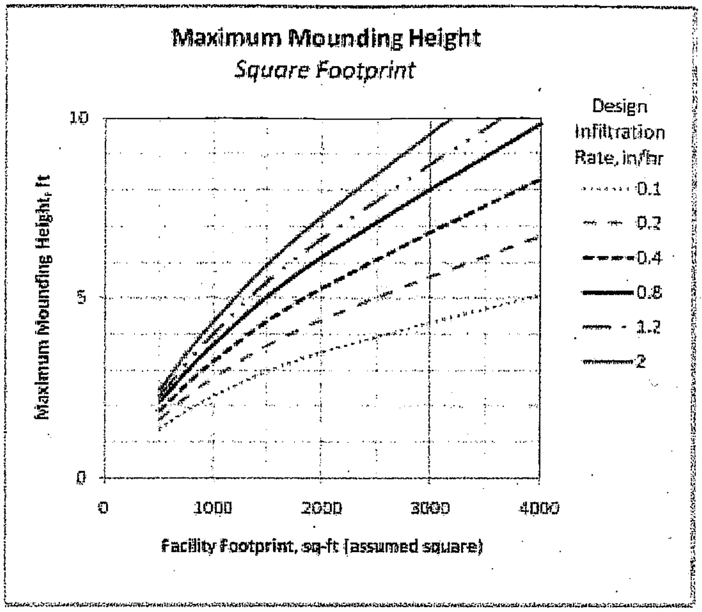
- Compute maximum mounding heights using USGS tool

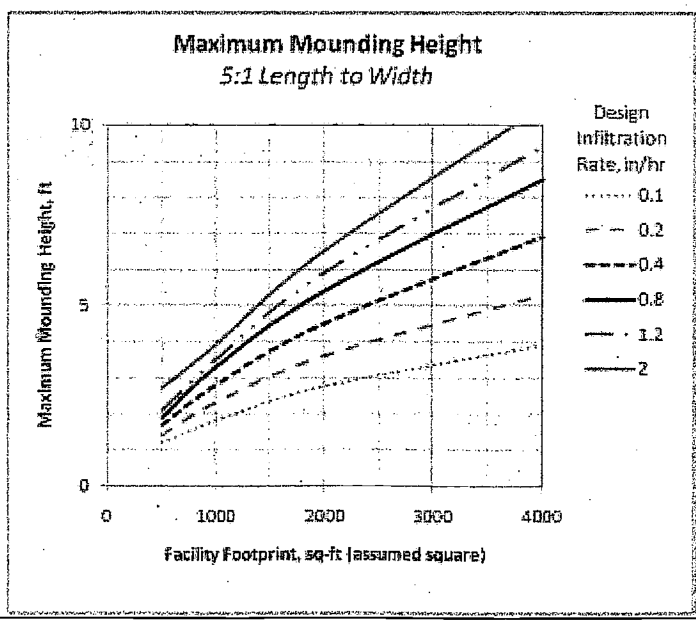
Solution:

Maximum mounding heights calculated with the USGS tool are given in Figure VIII.1. While these results reflect a relatively conservative case, they indicate that system size and design infiltration rate both influence the potential for mounding. In addition, a linear geometry reduces the magnitude of mounding somewhat compared to a square geometry with the same footprint.

Figure VIII.1: Example Calculations of Maximum Mounding Height by Facility Configuration from USGS Calculator (Carlton, 2010)

(For illustration purposes only based on input assumptions above; inputs shall be based on professional judgment)





VIII.3. Groundwater Plumes

Infiltration shall not be allowed in the vicinity of mapped or potential groundwater plumes, except where infiltration would not adversely impact groundwater conditions as determined via a site-specific or watershed study applicable to the site. In the absence of a site specific study, the following criteria apply:

- Infiltration is prohibited within *plume protection boundaries* identified by Orange County Water District (OCWD) (See **Figure VIII.2**), or equivalent boundaries identified by applicable groundwater agencies, unless a site specific study demonstrates that infiltration would not adversely impact groundwater conditions.
- Infiltration is prohibited in identified natural pollutant source areas (e.g., selenium) (See **Figure VIII.2**), unless a site specific study demonstrates that infiltration would not adversely impact groundwater conditions,
- Infiltration is prohibited within 250 feet of contaminated sites, such as sites found in the Geotracker or EviroStor databases (**http://geotracker.swrcb.ca.gov/**, **http://www.envirostor.dtsc.ca.gov/public/**), unless a site specific study demonstrates that infiltration would not adversely impact groundwater conditions. The study must include a review of the magnitude and type of the original contaminants and byproducts shall be used to assess the level of risk posed by infiltration in the vicinity of closed sites. This criterion applies to active contaminated sites or closed sites that have significant remaining potential for pollutant mobilization as a result of stormwater infiltration.
- A site-specific investigation shall always be performed to assess the feasibility of stormwater infiltration when the project proposes to redevelop a previously-contaminated site (e.g., Brownfields or otherwise contaminated).

As locations, boundaries, and number of contamination sites is subject to change, it is the responsibility of applicants to use the most up-to-date maps available from the permittees and applicable groundwater management agencies. Requirements for conducting site-specific studies vary with project size and are identified in Section VIII.8.

Basis for 250-foot Setback

The 250-foot separation distance from contaminated sites is based on the following considerations:

- In general terms, the degree of subsurface contamination typically decreases in the horizontal direction away from a contaminated site (although there can be site-specific conditions where this is not the case);
- As the distance between a contaminated site and a potential engineered infiltration system increases, the risk decreases that the engineered infiltration system will infiltrate water into subsurface contamination or otherwise negatively affect contamination originating from the contaminated site;
- By precluding engineered infiltration systems within 250 feet of a contaminated site, the risk decreases that infiltration would be increased through an area of the subsurface containing non-aqueous phase liquid contamination or areas with groundwater containing very high levels of contamination;
- A survey of sites contaminated with petroleum-related products estimated horizontal benzene plume lengths (California Leaking Underground Fuel Tank (LUFT) Historical

Case Analysis, UCRL-AR-122207, prepared by Lawrence Livermore National Laboratory, 1995). Based on a 10 part per billion concentration threshold, the survey estimated that 90 percent of the sites had benzene plume lengths of 261 feet or less. Some contaminants may have longer or shorter plume lengths than benzene and the amount of data on plume lengths is increasing as additional data are collected. Additional data and analysis may warrant reconsideration of this issue in the future.

VIII.4. Requirements for BMP Selection by Tributary Land Use Activities

Table VIII.1 provides criteria for selection of BMPs to address the potential for contamination of groundwater from tributary land use activities. Infiltration BMPs shall be selected and applied as recommended by **Table VIII.1**.

To prevent contamination from materials used in the construction of the infiltration BMP itself, soil media, construction materials, and construction practices should be appropriately selected to ensure that hazardous chemicals or groundwater pollutants of concern are not inadvertently leached to the underlying groundwater.

Figure VIII.2: North Orange County Groundwater Basin Protection Boundary and Plume Protection Boundaries (See Figure XVI.2f for high resolution exhibit)

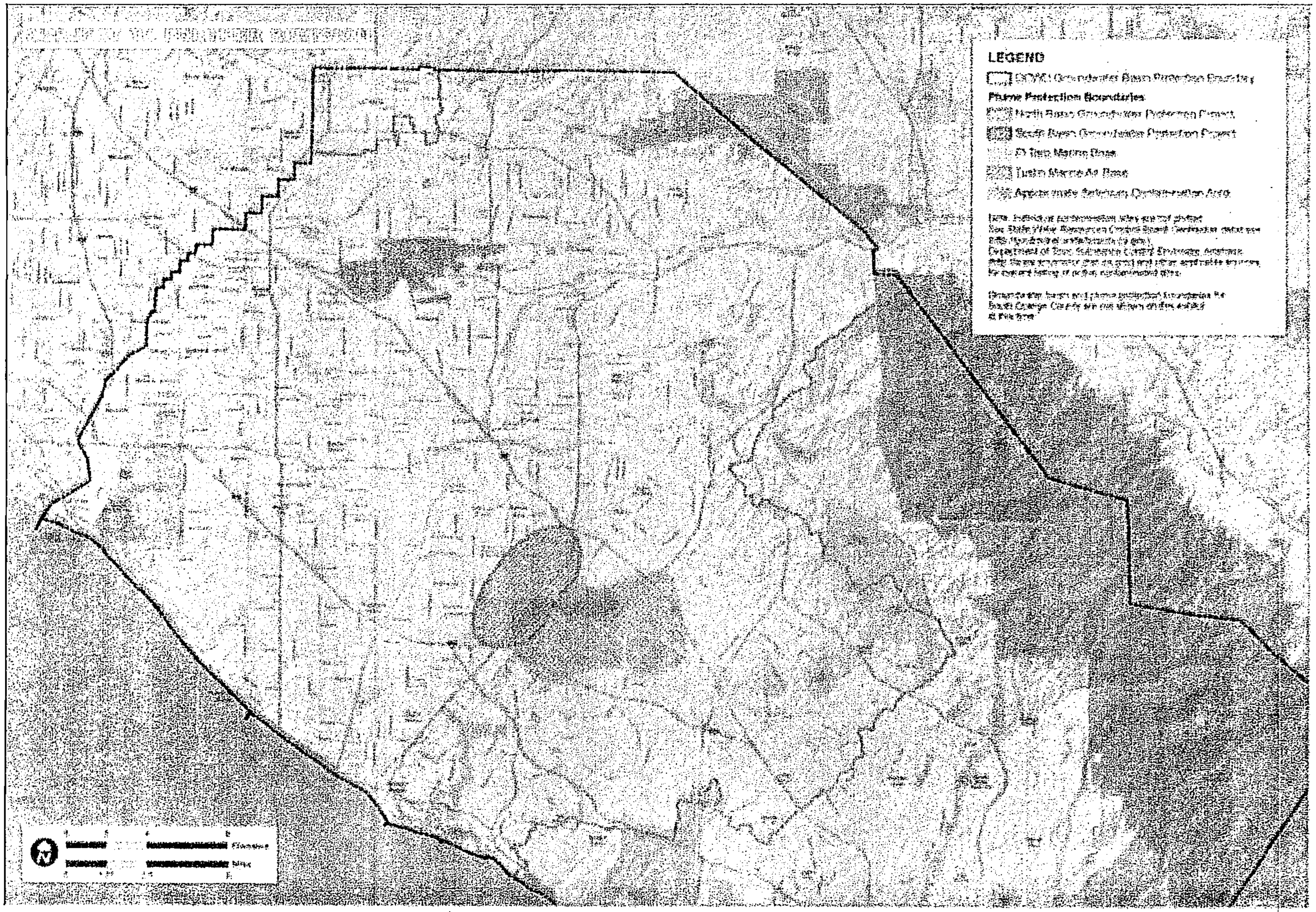


Table VIII.1: Recommendations/Requirements for BMP Selection to Minimize Groundwater Quality Impacts

Tributary Area Risk Category	Narrative Description of Category	Example Land Use Activities	BMP Selection Requirements
Low Runoff Contamination Potential	BMP receives runoff from a mix of land covers that are expected to have relatively clean runoff; significant spills in tributary area are unlikely.	• Rooftops with roofing material and downspouts free of copper and zinc • Patios, sidewalks, and other pedestrian areas • Mixed residential land uses with applicable source controls • Institutional land uses with applicable source controls • Driveways and minor streets	• Any infiltration BMP type may be used • Pretreatment for sediment is strongly recommended, as applicable, to mitigate clogging
Moderate Runoff Contamination Potential	BMP receives runoff from a mix of land covers, more than 10 percent of which have the potential to generate stormwater pollutants at levels that could potentially contaminate groundwater; there is potential for minor spills in the tributary area.	• Roadways greater than 5,000 ADT but less than 25,000 ADT • Commercial and institutional parking lots • Commercial land uses • Light industrial that does not include usage of chemicals that are mobile in stormwater and groundwater • Trash storage areas	• Any infiltration BMP type may be used • Pretreatment shall be used • The type of pretreatment shall be selected to address potential groundwater contaminants potentially found in stormwater runoff.
High Runoff Contamination Potential	BMP receives runoff from a mix of land covers, more than 10 percent of which have significant unavoidable potential to generate stormwater pollutants in quantities that could be detrimental to groundwater quality; and/or there is significant potential for major spills that could drain to BMPs.	• Roads greater than 25,000 ADT • Heavy and light industrial pollutant source areas, including areas with exposed industrial activity and high use industrial truck traffic, and any areas that cannot be isolated these areas. Does not include lower risk source sources areas within industrial zones (e.g., roofs, offices, and parking areas) that are hydrologically isolated from industrial pollutant source areas • Automotive repair shops • Car washes • Fleet storage areas • Nurseries, agriculture, and heavily managed landscape areas with extensive use of fertilizer • Fueling stations (infiltration prohibited under all conditions)	• Infiltration is prohibited unless advanced pretreatment and spill isolation can be feasibly used and enhanced monitoring and inspection are implemented. • Large projects[15] must evaluate feasibility of advanced pretreatment and spill isolation. • Small projects[15] may consider infiltration to be infeasible with narrative discussion.

[15] See Table VIII.2 for definition of "Large" and "Small" projects.

VIII.5. Well Head Protection and Septic Systems

To ensure protection of groundwater quality, the following criteria shall be met:

- Stormwater shall not be infiltrated within 100 feet horizontally of a water supply well, non-potable well, or spring.
- Stormwater shall not be infiltrated within 100 feet horizontally of a septic tank drain field.

Because data regarding the location of supply wells, springs, and septic systems is not generally available to the public, the project proponent is strongly encouraged to consult with the local review agency early in the WQMP preparation process to determine whether these conditions apply to all or part of the project site.

VIII.6. Stormwater Runoff Pollutants

Stormwater BMPs shall be selected to minimize the introduction of contaminants into groundwater via infiltration of stormwater runoff. The potential for groundwater contamination from pollutants found in stormwater runoff is a function of the land use activities that are present in the tributary area to the BMP. Table VIII.2 provides requirements for selection of BMPs and pretreatment devices based on the level of risk posed by land use activities.

VIII.7. Consultation with Applicable Groundwater Management Agencies

Projects that propose to infiltrate stormwater are required to consult with the applicable groundwater management agency to the extent necessary to ensure that groundwater quality is protected.

The process for consultation with applicable groundwater management agencies was under development at the time of publication and is not included in this TGD. It is anticipated that guidelines will be published in the future that include:

- Description of the consultation process
- Description of the conditions under which consultation is necessary
- Discussion of the point in the project process at which consultation should be initiated for qualifying projects
- Discussion of the review schedule and fees (if applicable)
- Materials that should be submitted as part of this process
- Discussion of potential outcomes and actions from this process

Until guidelines are published, all infiltration activities should be coordinated with the applicable groundwater management agency, such as OCWD, to ensure groundwater quality is protected. It is recommended that coordination be initiated as early as possible during the Preliminary/Conceptual WQMP development process.

Applicable groundwater management agencies

North Orange County Groundwater Basin:	Orange County Water District Attn: Director of Planning 18700 Ward Street Fountain Valley, CA 92708
San Juan Groundwater Basin:	San Juan Basin Authority *Contact info to be provided*

In addition, LID infiltration facilities may potentially be categorized as "Class V Injection Wells" under the federal Underground Injection Control (UIC) Program, which is regulated in California by U.S. EPA Region 9. The EPA defines a Class V well as any bored, drilled, or driven shaft, or dug hole that is deeper than its widest surface dimension, or an improved sinkhole, or a subsurface fluid distribution system (an infiltration system with piping to enhance infiltration capabilities). A UIC permit may be required for such a facility (for details see http://www.epa.gov/region9/water/groundwater/uic-classv.html).

VIII.8. Technical Requirements for Site Specific Study of Infiltration Impacts on Groundwater Quality

VIII.8.1. Project Size Applicability

Regardless of project size, any project proposing to use infiltration BMPs within a *plume protection boundary* (see Exhibit IX-3) or within 250 ft of a contaminated site shall conduct a site-specific study prior to using these BMPs to demonstrate that infiltration will not have adverse impacts on groundwater quality.

For small projects, a site-specific study is not required unless the project proponent chooses to use infiltration, in which case a site-specific study shall be prepared. If the proponent does not choose to use infiltration, the presence of one of the above-referenced conditions (including: shallow groundwater depth or mounding potential, presence of groundwater plumes, proximity to wellheads or septic systems, risks from land use activities, or other site-specific feasibility concerns) is sufficient to demonstrate infeasibility of infiltration BMPs.

For large projects, a site-specific study is required to determine if infiltration is feasible and would not adversely impact groundwater quality in the vicinity of plume(s) and/or contaminated sites, or adversely affect groundwater drinking supplies.

Large projects and small projects are defined in **Table VIII.2**.

Table VIII.2: Definition of Project Size Categories

	Residential	Commercial, Institutional	Industrial
Small Projects	Less than 10 acres and less than 30 DU	Less than 5 acres and less than 50,000 SF	Less than 2 acre and less than 20,000 SF
Large Projects	Greater than 10 acres or greater than 30 DU	Greater than 5 acres or greater than 50,000 SF	Greater than 2 acre or greater than 20,000 SF

VIII.8.2. Information and Documentation Required in Site-Specific Study

If a project proponent proposes to use infiltration BMPs within a *plume protection boundary* (see Exhibit IX-3) or within 250 ft of a contaminated site, the project proponent shall provide a written report to demonstrate that infiltration does not pose an adverse risk to groundwater. The written report should be prepared by a state-certified professional and provided to OCWD for review and comment. The report shall document that the following conditions are met:

1. Lateral and vertical extent of soil or groundwater contamination is defined at the site and is defined for off-site areas if contamination has migrated to the boundary of the site.
2. Groundwater conditions are defined based on site specific data (e.g., subsurface sediment characteristics, depth to groundwater, groundwater flow direction, rate of groundwater movement).
3. Ongoing monitoring of soil or groundwater contamination is occurring and will continue to occur, as necessary.
4. A state-certified professional evaluates soil and groundwater data and evaluates whether proposed stormwater infiltration could cause adverse impacts to groundwater quality; an adverse impact to groundwater quality could include changing the movement of groundwater contamination, causing additional amounts of contamination in the unsaturated zone to migrate into the saturated zone, or negatively impacting an existing remediation system.
5. The applicable regulatory agency is identified and has continuing authority to require additional investigation or cleanup work if stormwater infiltration causes an adverse impact on groundwater quality.

In summary, infiltration shall not be allowed for sites where there is substantial evidence of an adverse risk to groundwater quality.

Worksheet I: Summary of Groundwater-related Feasibility Criteria

1	Is project large or small? (as defined by Table VIII.2) circle one	Large		Small
2	What is the tributary area to the BMP?	A		acres
3	What type of BMP is proposed?			
4	What is the infiltrating surface area of the proposed BMP?	A_{BMP}		sq-ft
5	What land use activities are present in the tributary area (list all)			
6	What land use-based risk category is applicable?	L	M	H
7	If M or H, what pretreatment and source isolation BMPs have been considered and are proposed (describe all):			
8	What minimum separation to mounded seasonally high groundwater applies to the proposed BMP? See Section VIII.2 (circle one)	5 ft		10 ft
9	Provide rationale for selection of applicable minimum separation to seasonally high mounded groundwater:			
10	What is separation from the infiltrating surface to seasonally high groundwater?	SHGWT		ft
11	What is separation from the infiltrating surface to mounded seasonally high groundwater?	Mounded SHGWT		ft
12	Describe assumptions and methods used for mounding analysis:			
13	Is the site within a plume protection boundary (See Figure	Y	N	N/A

Worksheet I: Summary of Groundwater-related Feasibility Criteria

	VIII.2)?	
14	Is the site within a selenium source area or other natural plume area (See **Figure VIII.2**)?	Y N N/A
15	Is the site within 250 feet of a contaminated site?	Y N N/A
16	If site-specific study has been prepared, provide citation and briefly summarize relevant findings:	
17	Is the site within 100 feet of a water supply well, spring, septic system?	Y N N/A
18	Is infiltration feasible on the site relative to groundwater-related criteria?	Y N
Provide rationale for feasibility determination:		

Note: if a single criterion or group of criteria would render infiltration infeasible, it is not necessary to evaluate every question in this worksheet.

APPENDIX IX. TECHNICAL BASIS FOR GREEN ROOF DESIGN CRITERIA

The purpose of this appendix is to present minimum criteria for green roofs (roofs with growing media and vegetation) to be considered "self-retaining" for new development and significant redevelopment projects in Orange County. Self-retaining areas are designed to retain the DCV and no further management of these areas is required to meet LID and treatment control performance criteria. This category also includes brown roofs, which are designed with vegetation intended to go seasonally dormant during dry periods. This document describes the functional definition of "self-retaining" that has been applied to green roofs, presents an overview of the analytical methods used to evaluate performance of a range of design criteria, and presents the results of this analysis in terms of the minimum design criteria for green roofs to be considered self-retaining.

IX.1. Functional Definition of "Self-Retaining" for Green roofs

HSCs are group of low-tech stormwater management measures that reduce stormwater runoff volume through landscape dispersion and interception of stormwater. As described above, if an HSC is to be considered "self-retaining," it should fully retain the volume from the LID design storm event.

Green roofs are a form of HSC. These systems reduce stormwater runoff volume by retaining a portion of rainfall in soil pores and surface and plant depression storage during storm events and making it available for subsequent ET. Green roofs also provide biotreatment/ biofiltration of water draining through and over roofs, removing pollutants deposited from the atmosphere or from adjacent transportation land uses. Finally, green roofs can have additional benefits beyond stormwater management, including reductions in building heating and cooling costs and reductions in urban heat island effects. As such, green roofs should be encouraged where they can provide appreciable benefit for stormwater management. They do require irrigation, so their effects on water supply should be considered.

The volume reduction potential of green roofs is relatively limited in the southern California climate because of typical patterns of precipitation and ET: during winter months when the majority of rainfall occurs, and particularly during the typical short periods of back-to-back rainfall events, ET rates are relatively low, and pore space is recovered relatively slowly. As such, it is not generally possible for green roofs of a reasonable thickness to provide reliable reduction of the entire DCV within the timeframe criteria applied to other HSCs. To recognize this limitation and still encourage the use of these system, a green roof would be considered to be "self-retaining" (i.e., requiring no other stormwater mitigation measures for the DCV) if the roof retains at least 40 percent of average long term precipitation volume and biotreats the remaining volume.

IX.2. Analysis Inputs

To determine the minimum design criteria for a green roof to be considered self-retaining, a simple modeling analysis of precipitation, ET patterns, and green roof design parameters was conducted. This analysis included the following inputs:

- **60 year of hourly precipitation data** from the NCDC Los Angeles International Airport (LAX) climate station (COOP ID: 045114)[16]. The average annual precipitation at LAX is 12 inches, which is approximately the same as observed over much of Orange County, therefore this analysis is applicable to Orange County.
- **Monthly normal reference ET data** from the NCDC Cooperative Summary of the Day at LAX (COOP ID: 045114) (See note 16).
- **Ranges of green roof extensiveness.** Extensiveness is defined as the ratio of the area covered by green roof to the area tributary to the roof (including the roof itself). Extensiveness has a maximum of 1.0. For the study, extensiveness varied from 0.5 (half the roof occupied by green roof with the remaining area draining to the green roof) to 1.0 (the full roof covered by the green roof, or the green roof portion not receiving any "run-on" from other areas).
- **Ranges of landscape coefficients.** The landscape coefficient (K_L) is a multiplier on the ET rate that accounts for the plant species, micro climate (exposure, etc.), and the density of vegetative cover. For the study, landscape coefficients of 0.5 and 0.75 were evaluated, representing low water use species and moderate water use species, respectively. Landscape coefficients are generally believed to be higher on roof tops than for ground-level landscaping because of high exposure to sun and wind. It is not recommended that high water use species be used in green roofs because of the high irrigation demand exerted during summer months and winter dry periods.
- **Ranges of soil moisture retention depth.** Green roof moisture retention depth is the equivalent depth of water that a green roof can hold long enough for ET to have an appreciable effect. For engineered extensive or intensive roofs, this is defined as the field capacity (FC, the volumetric water content retained in soil after a prolonged period of draining) minus the wilting point (WP, the lowest volumetric water content that can be achieved via plant transpiration processes). This is generally 15 to 20 percent of the

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[16] This analysis was prepared from data originally developed for another Geosyntec project; therefore different input data sources have been used than were used for other analyses described in this TGD. The input data used for this analysis is believed to be representative of Orange County and differences are very likely within the range of model sensitivity/uncertainty.

actual thickness of the green roof, depending on the characteristics of the growing media. Some proprietary green roof systems utilized specialized light weight media with enhanced soil moisture retention properties or synthetic materials such as plastic cup layers and wicking materials. These systems are generally specified in terms of the effective depth of water they retain (i.e., the soil moisture retention depth). Soil moisture retention depth was varied from 0 up to 4 inches for this study, representing simple green roofs up to approximately 30 inches deep.

IX.3. Analysis Methods

For the purpose of this analysis, Geosyntec developed a model written in VBA (Excel) that incorporates the inputs described above on an hourly basis and tracks the transient storage contained in soil moisture storage. The model can best be thought of as physically representing a bucket of water, where the water level in the bucket corresponds to the amount of moisture held in the green roof soil. Precipitation is applied over the roof and other areas tributary to the roof at hourly time steps corresponding to historical records. When the capacity of the soil moisture layer is exceed, runoff occurs. During and between events, the monthly normal ET rate is applied to the stored water to recover the storage in the soil moisture layer (i.e., empty the bucket). The precipitation and runoff is tracked and totaled for the model run, yielding the average fraction volume removed.

IX.4. Results

Results are presented in terms of the soil moisture retention depth required to achieve at least 40 percent reduction in volume. Results are presented in Table IX.1. Graphical output of model results are shown in Figure IX.1 and Figure IX.2, and are expressed in terms of landscape coefficient. The landscape coefficient describes the fraction of reference ET that can be assumed to be evapotranspired for a given plant palette. The higher the landscape coefficient, the shallower the depth of the green roof needs to be to achieve 40 percent retention. This would be expected, since water lost to ET is retained (does not run off) and higher landscape coefficient increases the rate of ET. Likewise increasing the extensiveness of a roof has the same effect, since larger green roof surface area per unit of stored volume yields faster moisture recovery rates.

It should be noted that when designing a green roof, consideration should be given to summer irrigation demands as well as wet season performance. While a higher landscape coefficient and more extensive area would theoretically increase wet season performance, this would also tend to increase irrigation demand during the dry season and during dry periods of the wet season.

Table IX.1: Green Roof Moisture Retention Depth Required for 40 Percent Volume Reduction, Los Angeles/Orange County

Landscape Coefficient (K_L) = 0.5						
Extensiveness	0.5	0.6	0.7	0.8	0.9	1.0
Minimum Required Moisture Retention Depth, inches	1.3	1.05	0.9	0.8	0.7	0.6
Typical Soil Depth Required to Provide Minimum Moisture Retention Depth(FC - WP = 0.15)	8.7	7.0	6.0	5.3	4.7	4.0
Landscape Coefficient (K_L) = 0.75						
Extensiveness	0.5	0.6	0.7	0.8	0.9	1.0
Minimum Required Moisture Retention Depth, inches	0.9	0.75	0.65	0.55	0.5	0.45
Typical Soil Depth Required to Provide Minimum Moisture Retention Depth(FC - WP = 0.15)	6.0	5.0	4.3	3.7	3.3	3.0

K_L = Landscape Coefficient; WP = soil wilting point; FC = soil field capacity

Figure IX.1: Green Roof Performance Relationships for Los Angeles and Orange County, Landscape Coefficient (KL) = 0.5 (Low water use plant palette)

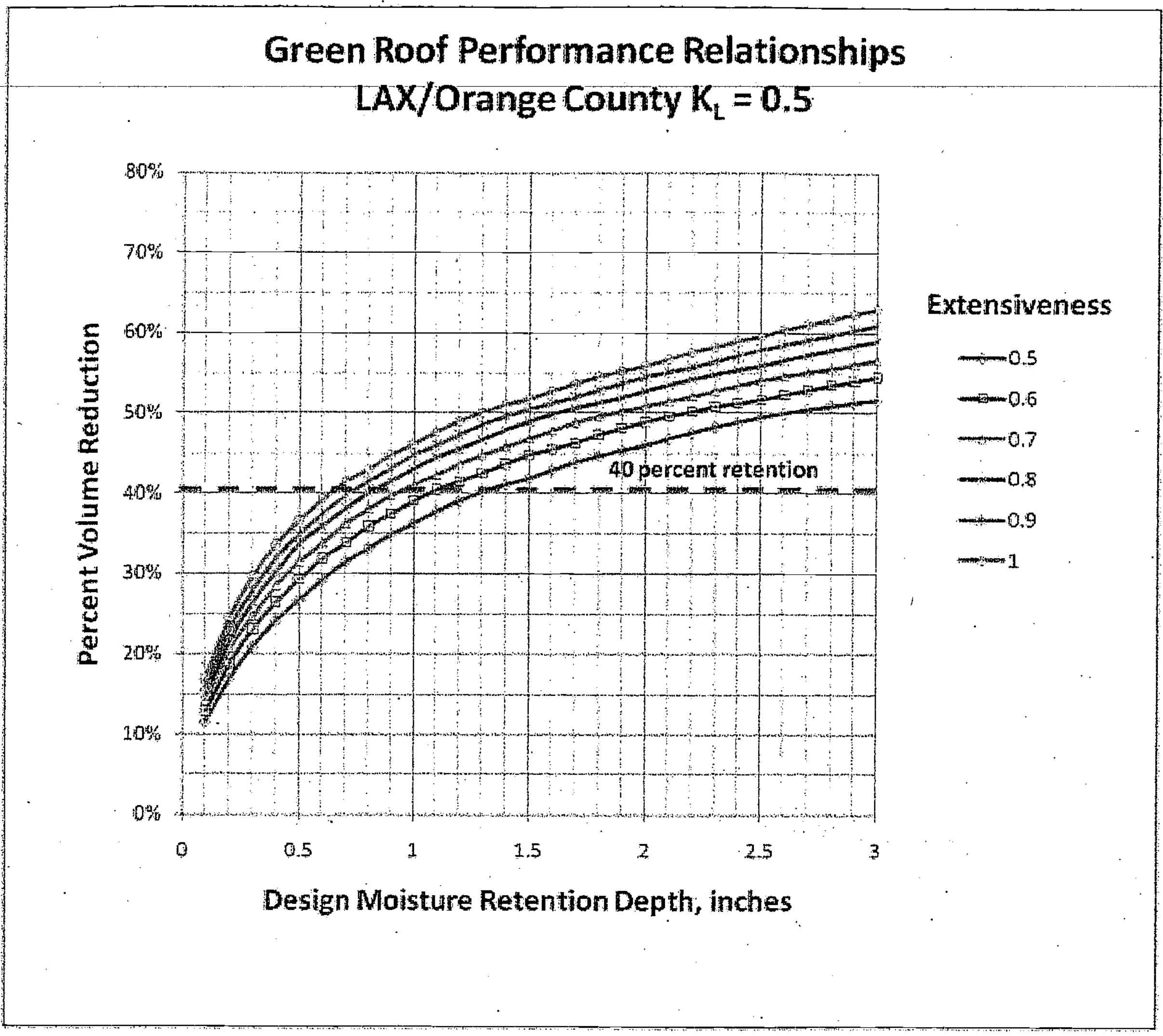
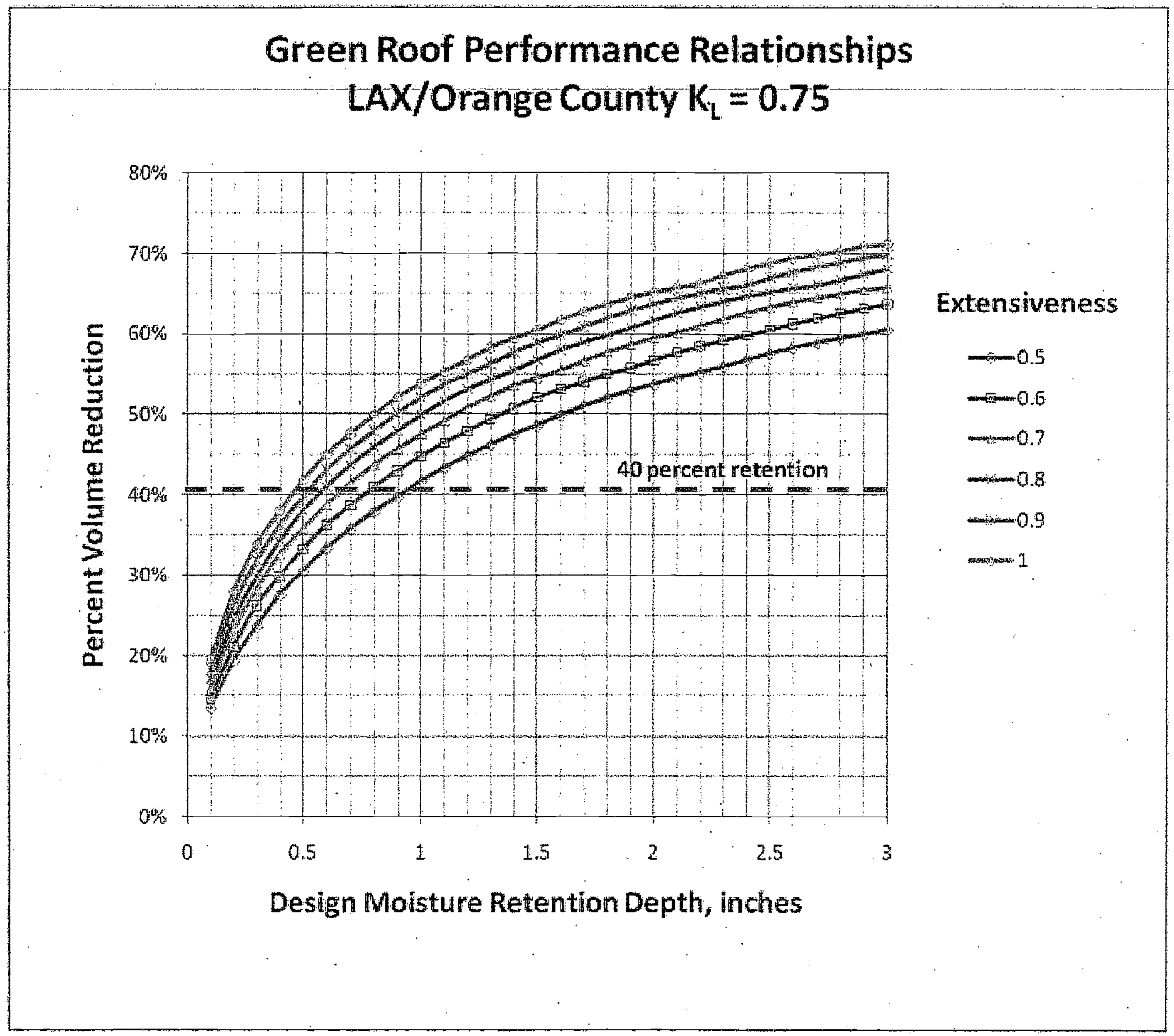


Figure IX.2: Green Roof Performance Relationships for Los Angeles and Orange County, Landscape Coefficient (K_L) = 0.75 (Moderate water use plant palette)



APPENDIX X. HARVEST AND USE DEMAND CALCULATIONS AND FEASIBILITY SCREENING

X.1. Introduction

The purpose of this appendix is to provide guidance for calculating harvested water demand and provide the technical basis for the harvest and use feasibility screening thresholds. This appendix contains the following:

- References for harvested water demand and guidance for preparing project-specific harvested water demand calculations
- Evaluation of required harvested water demand for minimum partial feasibility of harvest and use systems

Harvested water demand should be evaluated at the scale of the project, and not limited to single drainage areas. It is assumed that harvested water collected from one drainage area could be used within another.

X.2. Harvested Water Demand Calculation

The following sections provide technical references and guidance for estimating the harvested water demand of a project. These references are intended to be used for the planning phase of a project and for feasibility screening purposes.

X.2.1. Key Differences in Demand Calculations for Harvest and Use Feasibility versus Water Supply Planning

It is very important to note that harvested water demand calculations differ in purpose and methods from water demand calculations done for water supply planning. When designing harvest and use systems for stormwater management, a reliable method of relatively quickly regenerating storage capacity (i.e., using water) must exist to provide storage capacity for subsequent storms. Therefore, demand calculations for harvest and use BMPs should attempt to estimate the *actual demand that is reliably present to drain stormwater cisterns during the wet season and especially within short-term (week to a couple of weeks) series of storms that are typical.* This objective is fundamentally different from the objectives of water demand forecasting calculations done for water supply planning, which may err toward higher estimates of demand to provide conservatism to account for uncertainty. Harvested water demand calculations used to determine the feasibility of harvest and use BMPs must be based on estimates of actual expected demand that are reliably present to drain the cistern during the wet season.

X.2.2. Types of Harvested Water Demand

Types of non-potable water demand anticipated to be applicable in the foreseeable future include:

- Toilet and urinal flushing
- Irrigation
- Vehicle washing
- Evaporative cooling
- Dilution water for recycled water systems
- Industrial processes
- Other non-potable uses

The following sections are divided between toilet flushing, outdoor irrigation demand, and other non-potable demands. The primary distinction between toilet/urinal flushing and irrigation demand is the level of treatment and disinfection that is required to use the water and the seasonal pattern of the demand. Other non-potable demands (e.g. industrial processes for example) are anticipated to be highly project specific and should be calculated using project-specific information.

X.2.3. Toilet and Urinal Flushing Demand Calculations

The following guidelines should be followed for computing harvested water demand from toilet and urinal flushing:

- If reclaimed water is planned for use for toilet and urinal flushing, then the demand for harvested stormwater is equivalent to the total demand minus the reclaimed water supplied, and should be reduced by the amount of reclaimed water that is available during the wet season. The basis for this priority is provided in Section **X.2.8**.
- Demand calculations for toilet and urinal flushing should be based on the average rate during the wet season for a typical year.
- Demand calculations should include changes in occupancy over weekends and around holidays and changes in attendance/enrollment over school vacation periods.
- For facilities with generally high demand but periodic shut downs (e.g., for vacations, maintenance, or other reasons), a project specific analysis should be conducted to determine whether performance stormwater management can be maintained despite shut downs.
- Such an analysis should consider the statistical distributions of precipitation and demand, foremost the relationship of demand to the wet seasons of the year.

Table X.1 provides planning level estimated toilet and urinal flushing demand per resident or employee for a variety of project types. The per capita use per day is based on daily employee or resident usage. For non-residential types of development, the "visitor factor" and "student factor" (for schools) should be multiplied by the employee use to account for toilet and urinal usage for non-employees using facilities.

Table X.1: Toilet and Urinal Water Usage per Resident or Employee

Land Use Type	Toilet User Unit of Normalization	Per Capita Use per Day		Visitor Factor[4]	Water Efficiency Factor	Total Use
		Toilet Flushing[1,2]	Urinals[3]			
Residential	Resident	18.5	NA	NA	0.5	9.3
Office	Employee (non-visitor)	9.0	2.27	1.1	0.5	7 (avg)
Retail	Employee (non-visitor)	9.0	2.11	1.4	0.5	
Schools	Employee (non-student)	6.7	3.5	6.4	0.5	33
Various Industrial Uses (excludes process water)	Employee (non-visitor)	9.0	2	1	0.5	5.5

1- Based on American Waterworks Association Research Foundation,1999. Residential End Uses of Water. Denver, CO: AWWARF

2 - Based on use of 3.45 gallons per flush and average number of per employee flushes per subsector, Table D-1 for MWD (Pacific Institute, 2003).

3 - Based on use of 1.6 gallons per flush, Table D-4 and average number of per employee flushes per subsector, Appendix D (Pacific Institute, 2003)

4 - Multiplied by the demand for toilet and urinal flushing for the project to account for visitors. Based on proportion of annual use allocated to visitors and others (includes students for schools; about 5 students per employee) for each subsector in Table D-1 and D-4 (Pacific Institute, 2003)

5 – Accounts for requirements to use ultra low flush toilets in new development projects; assumed that requirements will reduce toilet and urinal flushing demand by half on average compared to literature estimates. Ultra low flush (ULF) toilets are required in all new construction in California as of January 1, 1992. ULF toilets must use no more than 1.6 gallons per flush (gpf) and ULF urinals must use no more than 1 gpf. (http://www.fypower.org/com/tools/products_results.html?id=100139) Note: If zero flush urinals are being used, adjust accordingly.

X.2.4. General Requirements for Irrigation Demand Calculations

The following guidelines should be followed for computing harvested water demand from landscape:

- If reclaimed water is planned for use for landscape irrigation, then the demand for harvested stormwater should be reduced by the amount of reclaimed water that is available during the wet season. The basis for this priority is provided in Section X.2.8.
- Irrigation rates should be based on the irrigation demand exerted by the types of landscaping that are proposed for the project, with consideration for water conservation requirements.
- Irrigation rates should be estimated to reflect the average wet season rates (defined as November through April) accounting for the effect of storm events in offsetting harvested water demand. In the absence of a detailed demand study, it should be assumed that irrigation demand is not present during days with greater than 0.1 inches of rain and the subsequent 3 day period. This irrigation shutdown period is consistent with standard practice in land application of wastewater and is applicable to stormwater to prevent irrigation from resulting in dry weather runoff. Based on a statistical analysis of Orange County rainfall patterns, approximately 30 percent of wet season days would not have a demand for irrigation.
- If land application of stormwater is proposed (irrigation in excess of agronomic demand), then this BMP must be considered to be an infiltration BMP and feasibility screening for infiltration must be conducted. In addition, it must be demonstrated that land application would not result in greater quantities of runoff as a result of saturated soils at the beginning of storm events. Agronomic demand refers to the rate at which plants use water.

The following sections describe methods that should be used to calculate harvested water irrigation demand. While these methods are simplified, they provide a reasonable estimate of potential harvested water demand that is appropriate for feasibility analysis and project planning. These methods may be replaced by a more rigorous project-specific analysis that meets the intent of the criteria above.

X.2.5. OC Irrigation Code Demand Calculation Method

This method is based on the **County of Orange Landscape and Irrigation Code and Implementation Guidelines** Ordinance No. 09-010 (OC Irrigation Code). The OC Irrigation Code includes a formula for estimating a project's annual Estimated Applied Water Use (EAWU) based on the reference evaporation, landscape coefficient, and irrigation efficiency.

For the purpose of calculating harvested water irrigation demand applicable to the sizing of harvest and use systems, the EAWU has been modified to reflect typical wet-season irrigation demand. This method assumes that the wet season is defined as November through April. This method further assumes that no irrigation water will be applied during days with precipitation totals greater than 0.1 inches or within the 3 days following such an event. Based on these assumptions and an analysis of Irvine precipitation patterns, irrigation would not be applied during approximately 30 percent of days from November through April.

The following equation is used to calculate the Modified EAWU:

$$\text{Modified EAWU} = (ETo_{Wet} \times K_L \times LA \times 0.015) / IE$$

Where:

Modified EAWU = estimated daily average water usage during wet season
ETo_{Wet} = Average Reference ET from November through April (inches per month, See Section X.2.5.1)
K_L = Landscape Coefficient, $K_L = K_s \times K_d \times K_{mc}$ (See Section X.2.5.2)
K_s = species factor
K_d = density factor
K_{mc} = microclimate factor
LA = Landscape Area (sq-ft)
IE = Irrigation Efficiency (assume 90 percent for demand calculations)

In this equation, the coefficient (0.015) accounts for unit conversions and shut down of irrigation during and for the three days following a significant precipitation event:

0.015 = (1 mo/30 days)×(1 ft/12 in)×(7.48 gal/cu-ft)×(approximately 7 out of 10 days with irrigation demand from November through April)

When using this method, the worksheets contained within the OC Irrigation Code may be useful to determine the irrigation use for a project site, with the appropriate modifications to reflect the Modified EAWU calculations. These worksheets allow the user to area-weight the inputs for irrigation.

X.2.5.1. Reference ET Data

Table X.2contains data derived from CIMIS for the cities of Irvine, Santa Ana, and Laguna Beach.

Table X.2: Monthly Reference ET Rates for Orange County (Inches)

Station	J	F	M	A	M	J	J	A	S	O	N	D	Annual	Wet Season Average (in/mo) (Nov to Apr)
Irvine	2.2	2.5	3.7	4.7	5.2	5.9	6.3	6.2	4.6	3.7	2.6	2.3	49.9	3.00
Laguna Beach	2.2	2.7	3.4	3.8	4.6	4.6	4.9	4.9	4.4	3.4	2.4	2.0	43.3	2.75
Santa Ana	2.2	2.7	3.7	4.5	4.6	5.4	6.2	6.1	4.7	3.7	2.5	2.0	48.3	2.93

Source: County of Orange Landscape and Irrigation Code and Implementation Guidelines

X.2.5.2. Landscape Coefficient (K_L)

The Water Use Classifications of Landscape Species (WUCOLS, University of California and Department of Water Resources, 2000) should be used to determine the landscape coefficient that is applicable to each landscape irrigation zone. The landscape coefficient, K_L, is based on the product of the species factor (K_s), the density (K_d), and the microclimate (K_{mc}).

- The species factor is based on plant water needs derived from available data. At the time of the 2000 WUCOLs, 1,800 plant species had been evaluated for relative water needs. Specific species factors for these plant species are available in WUCOLs.
- The density factor is related to the vegetative or leaf cover for different plantings. Thinner or thicker than average density conditions are assigned density coefficients less than or greater than 1.0, respectively.
- The microclimate factor is related to features present in the urban landscape that influence temperature, wind, shading, and other climatic factors. An 'average' microclimate is equivalent to reference ET conditions (1.0), which is relatively uninfluenced by nearby buildings, structures, etc.

Table X.3 provides a general overview of these factors, ranging from low to high water use plant palettes.

Table X.3: Species, Density, and Microclimate Factors from WUCOLs for High, Moderate, Low and Very Low Water Use Plant Palettes

	High	Moderate	Low	Very Low
Species Factor* (k_s)	0.7-0.9	0.4-0.6	0.1-0.3	<0.1
Density (k_d)	1.1-1.3	1.0	0.5-0.9	
Microclimate (k_{mc})	1.1-1.4	1.0	0.5-0.9	

Source: Water Use Classifications of Landscape Species (WUCOLS, University of California and Department of Water Resources, 2000)

Table X.4 provides recommended composite landscape coefficients that are appropriate for planning purposes and feasibility screening.

Table X.4: Planning Level Recommendations for Landscape Coefficient (KL)

General Landscape Type	Recommended Planning Level Landscape Coefficient (K_L)
Conservation Landscape Design (non-active turf)	$K_L = 0.35$
Active Turf Areas	$K_L = 0.7$

X.2.5.3. Planning Level Irrigation Demands

Using the inputs above, daily average wet season demands were developed for an acre of irrigated area based on location and landscape type (Table X.5). These demand estimates can be used to calculate the drawdown of harvest and use systems for the purpose of LID BMP sizing calculations (**Appendix I**).

Table X.5: Modified EWUA Daily Average Irrigation Demand by Location and Landscape Coefficient

	Daily Average Modified EWUA (gpd per irrigated acre)		
General Landscape Type	Irvine	Santa Ana	Laguna
Conservation Landscape Design (non-active turf): $K_L = 0.35$	740	720	680
Active Turf Areas: $K_L = 0.7$	1,480	1,450	1,360

X.2.6. EIATA Demand Calculation and Sizing Method

The TGD also supports an alternative approach for quantifying harvested water demand that relies on the Effective Irrigated Area to Tributary Area (EIATA) ratio as a tool for sizing stormwater harvest and use systems. This ratio was developed to be a primary indicator of the ability of a harvest and use system to effectively capture and manage stormwater.

The EIATA ratio is calculated as follows:

$$EIATA = LA \times K_L / [IE \times \text{Tributary Impervious Area}]$$

Where:

EIATA = effective irrigated area to tributary area ratio (ac/ac)
LA = landscape area irrigated with harvested water, sq-ft
K_L = Area-weighted landscape coefficient (per guidance above)
IE = irrigation efficiency (assume 0.90)

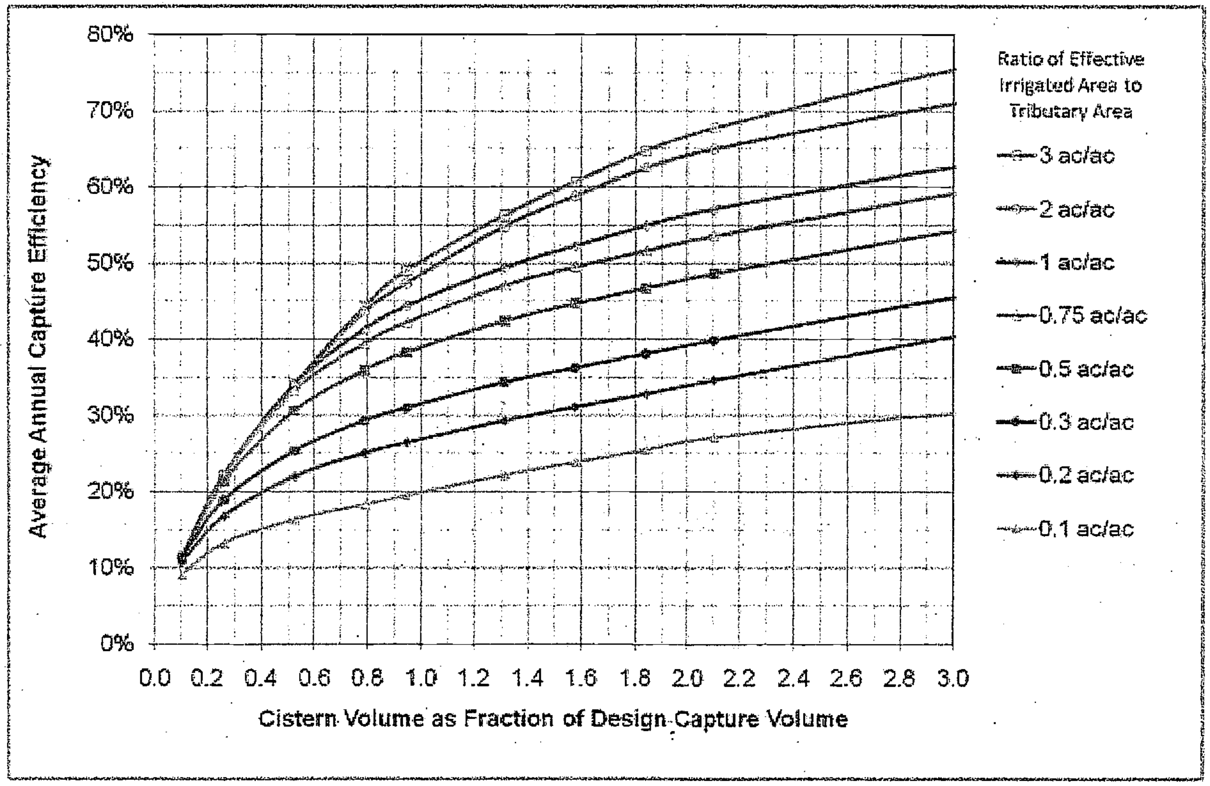
The calculated EIATA ratio can be used in

Figure X.1 to relate DCV to system performance.

Figure X.1 was developed in USEPA SWMM5.0 with 22 years of hourly precipitation and reference ET data from the Irvine CIMIS gage. The model accounts for short term suspension of irrigation demand following storm events by applying irrigation only after 0.25 inches of reference ET had occurred since the end of rainfall. This nomograph is applicable across Orange County.

Instructions for using this nomograph are contained in (**Appendix I**).

Figure X.1: Harvest and Use Sizing Nomograph



X.2.7. Calculating Other Harvested Water Demands

Calculations of other harvested water demands should be based on the knowledge of land uses, industrial processes, and other factors that are project-specific. Demand should be calculated based on the following guidelines:

- Demand calculations should represent actual demand that is anticipated during the wet season (November through April).
- Sources of demand should only be included if they are reliably and consistently present during the wet season.
- Where demands are substantial but irregular, a more detailed analysis should be conducted based on a statistical analysis of anticipated demand and precipitation patterns.

X.2.8. Reclaimed Water Priority in Demand Calculations

If reclaimed water is available to meet or partially meet project non-potable water demands, the decision to use reclaimed water or harvested runoff water rests with the project proponent. If the project proponent elects to use reclaimed water or is required to use reclaimed water based on conditions placed on the project, then the demand for harvested water should be reduced by the amount of reclaimed water available. This criterion effectively allows the project proponent to consider harvest and use to be infeasible if sufficient reclaimed water supply is available to meet the project demand for harvested water.

This criterion intentionally prioritizes the use of reclaimed water over harvested water in cases where demand overlaps. The use of reclaimed water is being prioritized based upon the following considerations:

- In Order 2009-06, the State Water Board finds that "…recycled water is safe for approved uses, and strongly supports recycled water as a safe alternative to potable water for such approved uses." There are several other state mandates for reduction of potable water demand.
- A substantial investment has been made in the production and distribution of reclaimed water by local agencies to reduce potable water demand to meet state mandates.
- Utilizing reclaimed water where available inherently reduces the amount of treated municipal effluent discharged to the ocean. For those entities that rely primarily on use of reclaimed water for disposal of treated wastewaters, such as the Irvine Ranch Water District, prioritizing use of runoff over reclaimed water could increase wastewater discharges significantly during wet weather periods.
- Utilizing the capacity of the reclaimed water system, where available, has a significantly larger benefit for offsetting potable water supply than stormwater harvest and use systems. Reclaimed water is available year round therefore can effectively fulfill all project non-potable water demands. In contrast, a harvested water system designed for stormwater management would tend to make water available for a relatively minor fraction of the year (during storm events and for a relatively short period after), thereby meeting a substantially lower fraction of the project non-potable water demand.
- It is possible to engineer and deploy a combined reclaimed water/harvested stormwater non-potable use system. However, the costs of including both options would be much

higher than employing one or the other. In addition, the most difficult time for reclaimed water disposal is during extended wet periods (irrigation reduced and more wastewater from inflow and infiltration).

- The State Board has evaluated the potential negative environmental consequences of reclaimed water on groundwater quality as part of developing its policy on reclaimed water, and the State Board supports the use of reclaimed water for landscape irrigation.
- It is noted that reclaimed water poses potential issues to groundwater quality, particularly salt and nutrient accumulation, which must be managed by providers of reclaimed water[17]. The priority for use of reclaimed water expressed in this TGD does not conflict or interfere with the obligation of reclaimed water providers to manage the application of reclaimed water. If, as a groundwater quality management action, a reclaimed water provider must limit the application of reclaimed water, it would be the responsibility of the reclaimed water provider to limit the amount of reclaimed water that is made available to a proposed project and/or limit its allowable uses on a project. This would limit the amount of project demand that can be offset by reclaimed water and would thereby require harvested water to be considered in applicable scenarios.

X.3. Planning Level Harvest and Use Feasibility Thresholds

This section describes the technical analysis and assumptions that were used to develop planning level feasibility thresholds for harvest and use systems. The intent of these thresholds is to identify projects with low potential for successful harvest and use and provide a means for applicants to readily demonstrate infeasibility of harvest and use, where clearly infeasible, without the need for a detailed project specific analysis.

X.3.1. Minimum Partial Capture Threshold

If a harvest and use system is designed with storage volume equal to the DCV from the tributary area but still achieves less than 40 percent capture, the system does not meet the minimum incremental benefit required to mandate its use (See discussion of threshold incremental benefit in **Appendix XIII**). This level of performance is termed the "minimum partial capture." A harvest and use system would be considered to achieve less than "minimum partial capture" if:

- Based on a system sized for the full DCV from the tributary area, and
- Based on the combined project demand for harvested water,
- The system draws down in greater than 30 days (720 hours), therefore captures less than 40 percent of average annual runoff (See Figure III.2).

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[17] In Water Quality Order No. 2000-07, the State Water Board determined that a Producer (i.e., reclaimed water purveyor) cannot shift responsibility for discharged salt to the User (i.e., project proponent).

Harvest and use systems with demand lower than required to achieve minimum partial capture are not required to be considered to demonstrate retention of stormwater to the MEP. If this is the case, other LID BMPs must be evaluated for retention and/or biotreatment of the Project DCV.

X.3.2. Demand Thresholds for Minimum Partial Capture

Table X.6 provides the minimum combined project demand to meet the minimum partial capture for the range of precipitation zones found in Orange County. Projects with a total demand below this value not required to prepare a project specific evaluation of harvest and use feasibility.

Table X.6: Harvested Water Demand Thresholds for Minimum Partial Capture

Design Capture Storm Depth[1], inches	Wet Season Demand Required for Minimum Partial Capture[2], gpd per impervious acre
0.60	490
0.65	530
0.70	570
0.75	610
0.80	650
0.85	690
0.90	730
0.95	770
1.00	810

1 - Based on isopluvial map (See **XVI.1**)
2 -Minimum Partial Capture is a performance standard whereby system performance exceeds 40 percent capture (See **Appendix XIII**) , such that the system must be considered for use even if it cannot achieve the full DCV.

X.3.3. TUTIA Ratio Thresholds for Minimum Partial Capture

Table X.7 provides thresholds for TUTIA (Toilet Users to Impervious Area) ratio required to achieve minimum partial capture of the stormwater DCV (i.e. at least 40 percent average annual capture efficiency with a system sized for the DCV). Projects with TUTIA ratios below this value and without other significant demands for harvested water are not required to prepare a project specific evaluation of harvest and use feasibility. The values in Table X.7 reflect the minimum TUTIA ratio required to achieve at least 40 percent average annual capture efficiency with a system sized for the DCV.

Table X.7: Minimum TUTIA for Minimum Partial Capture

Project Type	Residential	Retail and Office Commercial	Industrial	Schools[1]
Basis of Toilet User Calculation	*Resident*	*Employee (non-visitor)*	*Employee (non-visitor)*	*Employee (non-student)*
Design Capture Storm Depth, inches	Minimum TUTIA Ratio Required for Minimum Partial Capture (toilet users/impervious acre)			
0.6	74	98	125	21
0.65	80	106	135	23
0.7	86	114	145	24
0.75	92	122	155	26
0.8	98	130	165	28
0.85	104	138	176	30
0.9	110	146	186	31
0.95	117	154	196	33
1	123	162	206	35

1 – based on employees only; assumes approximately 5 students per employee.

X.3.4. Irrigated Area Thresholds for Minimum Partial Capture

Table X.8 provides thresholds for irrigated area per impervious acre for minimum partial capture of the stormwater DCV. Projects with irrigation area below this value and without other sources of significant demand will generally not be required to prepare a project specific evaluation of harvest and use feasibility. The values in Table X.8 reflect the minimum irrigated area per impervious area required to achieve at least 40 percent average annual capture efficiency with a system sized for the DCV.

Table X.8: Minimum Irrigated Area for Potential Partial Capture Feasibility

General Landscape Type	Conservation Design: $K_L = 0.35$			Active Turf Areas: $K_L = 0.7$		
Closest ET Station	*Irvine*	*Santa Ana*	*Laguna*	*Irvine*	*Santa Ana*	*Laguna*
Design Capture Storm Depth, inches	Minimum Required Irrigated Area per Tributary Impervious Acre for Potential Partial Capture, ac/ac					
0.60	0.66	0.68	0.72	0.33	0.34	0.36
0.65	0.72	0.73	0.78	0.36	0.37	0.39
0.70	0.77	0.79	0.84	0.39	0.39	0.42
0.75	0.83	0.84	0.90	0.41	0.42	0.45
0.80	0.88	0.90	0.96	0.44	0.45	0.48
0.85	0.93	0.95	1.02	0.47	0.48	0.51
0.90	0.99	1.01	1.08	0.49	0.51	0.54
0.95	1.04	1.07	1.14	0.52	0.53	0.57
1.00	1.10	1.12	1.20	0.55	0.56	0.60

Worksheet J: Summary of Harvested Water Demand and Feasibility

1	What demands for harvested water exist in the tributary area (check all that apply):			
2	Toilet and urinal flushing		☐	
3	Landscape irrigation		☐	
4	Other:____________________		☐	
5	What is the design capture storm depth? (**Figure III.1**)	d		inches
6	What is the project size?	A		ac
7	What is the acreage of impervious area?	IA		ac
	For projects with both toilet flushing and indoor demand			
8	What is the minimum use required for partial capture? (**Table X.6**)			gpd
9	What is the project estimated minimum wet season total daily use?			gpd
10	Is partial capture potentially feasible? (Line 9 > Line 8?)			
	For projects with only toilet flushing demand			
11	What is the minimum TUTIA for partial capture? (**Table X.7**)			
12	What is the project estimated TUTIA?			

Worksheet J: Summary of Harvested Water Demand and Feasibility

13	Is partial capture potentially feasible? (Line 12 > Line 11?)		
	For projects with only irrigation demand		
14	What is the minimum irrigation area required based on conservation landscape design? (Table X.8)		ac
15	What is the proposed project irrigated area? (multiply conservation landscaping by 1; multiply active turf by 2)		ac
16	Is partial capture potentially feasible? (Line 15 > Line 14?)		
Provide supporting assumptions and citations for controlling demand calculation:			

APPENDIX XI. CRITERIA FOR DESIGNING BMPS TO ACHIEVE MAXIMUM FEASIBLE RETENTION AND BIOTREATMENT

XI.1. Purpose and Intended Use

The purposes of this appendix are two-fold:

1) To provide guidance for designing biotreatment BMPs to achieve the maximum feasible Infiltration and ET. Where biotreatment BMPs are used, they must be designed to achieve this objective.
2) To provide guidance for designing BMPs to retain and biotreat stormwater to the maximum extent practicable (MEP) for sites that cannot fully retain or biotreat the DCV. Retention must be used to the MEP before biotreatment is used.

This section includes:

- Criteria for designing biotreatment BMPs to achieve maximum feasible infiltration and ET
- Criteria for designing BMPs to achieve maximum feasible retention of the stormwater design volume
- Criteria for designing BMPs to achieve maximum feasible retention plus biotreatment of the stormwater design volume
- Supporting criteria for designing BMPs to achieved maximum feasible retention plus biotreatment of the stormwater design volume

This Appendix is intended to be applied as referenced from the BMP selection and design process described in **TGD Section 2.4**.

XI.2. Criteria for Designing Biotreatment BMPs to Achieve Maximum Feasible Infiltration and ET

Infiltration and ET are volume reduction processes that occur in biotreatment BMPs, but they are not the principal treatment mechanism. However, these incidental processes must be promoted whenever biotreatment BMPs are designed for a project. This section is intended to be used design biotreatment to BMPs to result in maximum feasible infiltration and ET in cases where neither infiltration nor harvest and use are feasible based on infiltration feasibility criteria contained in **TGD Section 2.4**, or where infiltration BMPs and/or harvest and use BMPs are partially feasible and biotreatment BMPs must be used for the remaining design volume.

Evapotranspiration. To design biotreatment BMPs to achieve maximum feasible ET, BMPs shall be designed with amended soils consistent with Biotreatment Selection, Design, and Maintenance Requirements contained in **Appendix XII**.

Infiltration. To design biotreatment BMPs to achieve the maximum feasible infiltration, retention volume shall be provided below the lowest surface discharge point. The amount of retention volume that shall be provided depends on the infiltration rate of the soil. This practice shall not be used where there is substantial evidence that infiltration would pose an unmitigated risk per the infiltration feasibility criteria contained in **TGD Section 2.4**.

In cases where incidental infiltration passes the feasibility criteria in **TGD Section 2.4**, the criteria for designing biotreatment BMPs to achieve the maximum feasible infiltration are as follows.

XI.2.1. BMPs with Underdrains

Retention volume shall be provided below the underdrains of the BMP per the following criteria:

- A gravel storage layer shall be installed below the invert elevation of the underdrains, as applicable.
- Rock should be assumed to have a porosity of 0.4 unless otherwise supported, and
- The depth of rock should be selected so that the underdrain layer empties in 48 hours.
- Where the infiltration rate of the underlying soil is not known, a rate of 0.1 in/hr shall be assumed, resulting in a gravel depth of 12 inches.

Example:

- Soil has a *measured infiltration rate* of 0.15 inches per hour and risk-based factors do not apply.
- Depth that can be infiltrated in 48 hours = 0.15 in/hr × 48 hours = 7.2 inches
- Depth of gravel to provide this depth of water = 7.2 inches / 0.4 = 18 inches.

XI.2.2. Swales and Filter Strips without Underdrains

Retention volume shall be provided below the lowest surface discharge of the BMP per the following criteria:

- Check dams and outlet controls shall be installed, as applicable, to retain water on the surface and amended soil.
- The storage depth shall be selected to drain in 24 hours.
- Where the infiltration rate of the underlying soil is not known, a surface ponding depth of 2 inches shall be used.
- Soils shall be amended to promote infiltration consistent with Biotreatment Selection, Design, and Maintenance Requirements contained in **Appendix XII**.

Example:

- Underlying has an estimated infiltration rate of 0.1 inches per hour (with soil amendments considered) and risk-based factors do not apply.
- Depth that can be infiltrated in 24 hours = 0.1 in/hr × 24 hours = 2.4 inches.

XI.2.3. Dry Extended Detention Basins

Soils shall be amended to promote subsurface storage and infiltration consistent with Biotreatment Selection, Design, and Maintenance Requirements contained in **Appendix XII.**

XI.2.4. Wet Ponds and Constructed Wetlands

Wet ponds and constructed wetlands achieve high pollutant removal efficiency, in part, by maintaining a permanent pool. These BMPs should not be designed to achieve volume reduction as a primary goal; however some incidental volume reduction is expected to occur.

XI.3. Criteria for Designing BMPs to Achieve Maximum Feasible Retention of the Stormwater Design Volume

The requirements of this section are intended to apply when the entire DCV cannot be feasibly retained, but retention of the stormwater design volume is potentially feasible per the infeasibility criteria contained in **TGD Section 2.4.** BMPs shall be designed to retain the stormwater design volume to the MEP by demonstrating that the applicable criteria in the following subsections are met.

XI.3.1. General Criteria

If at any time in this process, the stormwater design volume can be retained and drawn down in less than or equal to 48 hours, or the BMP is demonstrated to retain 80 percent of average annual stormwater runoff (per methods contained in **Appendix III.3.2**) and HCOCs are addressed (per methods contained in **Appendix IV** (North Orange County permit area) or **Appendix V** (South Orange County permit area)), the system does not need to be sized to manage any additional stormwater volume.

If after meeting the criteria contained in the following subsections, it is demonstrated that the resulting design would retain less 40 percent of average annual runoff volume on a drainage area basis, the BMP is not required to be used to demonstrate that BMPs have been designed to retain the design volume to the MEP. Instead, a biotreatment BMP must be used to the MEP and must be designed to provide maximum feasible infiltration and ET. See **Appendix XIII** for the technical basis of the 40 percent capture threshold criterion.

XI.3.2. Infiltration BMPs

This section provides criteria that shall be met to demonstrate that infiltration BMPs have been designed to retain stormwater design volume to the MEP.

- All applicable HSCs shall be provided except where they are mutually exclusive with each other or with LID BMPs. Mutual exclusivity may result from overlapping BMP footprints such that either would be potentially feasible by itself, but both could not be implemented; and
- Site design allowances for infiltration BMPs shall meet or exceed minimum site design criteria (See Section XI.5.1 for criteria) , and
- Using the infiltration area that meets the minimum site design criteria (Section XI.5.1), and using a design infiltration that meets the minimum criteria for feasibility evaluation (See Section XI.5.2), BMP retention depth has been selected such that:
    - The combined storage volume provided by HSCs and retention BMPs equals or exceeds the stormwater design volume, or
    - Retention depth provided in BMPs (volume contained below lowest design discharge elevation) equals or exceeds the depth that would draw down in 48 hours based on the design infiltration rate. (For example: if the design infiltration rate is 0.25 inches per hour, this criterion would be met by providing at least 12 inches of retention storage [0.5 in/hr × 48 hr]). Intent: The depth corresponding to 48-hr drawdown represents the point of diminishing returns with respect to additional volume for additional capture efficiency, or
    - Deeper depth may be provided, however additional volume would be required to compensate for longer drawdown time (**Appendix** III.3.2). Surface drawdown shall not exceed 96 hours because of vector issues. Drawdown time of subsurface storage may exceed 96 hours, however consideration should be given to maintenance activities and plant survival, as applicable, in selecting a maximum subsurface drawdown time.

XI.3.3. Harvest and Use BMPs

This section provides criteria that shall be met to demonstrate that harvest and use BMPs have been designed to retain stormwater design volume to the MEP.

- All applicable HSCs (**Appendix XIV.1**) shall be provided except where they are mutually exclusive. Mutual exclusivity may result from overlapping BMP footprints such that either would be potentially feasible by itself, but both could not be implemented, and
- The combined storage volume provided in HSCs and harvest and use BMP(s) equals or exceeds the DCV, and
- All applicable demand for harvested water has been considered per criteria contained in **Appendix X**).

XI.4. Criteria for Designing BMPs to Result in Maximum Feasible Retention plus Biotreatment of the Stormwater Design Volume

The requirements of this section are intended to apply when the entire stormwater design volume cannot be feasibly retained, and therefore biotreatment BMPs must be added to the system to manage the remaining stormwater design volume to the MEP. Adding biotreatment BMPs to a system that has already been designed for the maximum feasible retention may necessarily require some retention volume to be converted to biotreatment volume to result in a design that achieves the highest combined pollutant load reduction. This section is intended to be used after the maximum feasible retention volume has been calculated.

The following criteria that shall be met to demonstrate that biotreatment BMPs have been designed to retain stormwater design volume to the MEP

- Biotreatment components shall be added to treat runoff from a project's drainage area without reducing retention such that combined, biotreatment and retention BMPs capture and manage 80 percent of average annual runoff (See approaches for sizing of treatment trains and multi-part systems in **Appendix III.5**),

  OR

- A combination BMP or multi-part BMP incorporating both retention and biotreatment volume shall be provided that capture and manages (retains plus biotreats) at least 80 percent of average annual runoff, and no more than half of the maximum feasible retention volume computed in Section **XI.3** has been shifted to biotreatment.

Any stormwater design volume that remains after meeting these criteria shall be considered infeasible to retain or biotreat on-site and alternative compliance obligations shall be computed as described in **Appendix VI**.

XI.5. Supporting Criteria for Designing BMPs to Achieve Maximum Feasible Retention and Biotreatment

This section provides criteria to support the design of BMPs to retain and biotreat the stormwater design volume to the MEP. The requirements of this section are intended to apply only to projects demonstrating that BMPs have been designed to achieve the maximum retention and biotreatment per Sections **XI.3** and **XI.4**, respectively, as referenced from these sections.

XI.5.1. Criteria for Site Design to Allow BMPs

Project site designs shall be developed to allow BMPs to the MEP per the criteria contained in this section. This section is applicable as referenced from Sections **XI.3** and **XI.4**.

- At least the recommended portion of the site specified Table XI.1 (or a more stringent table developed by local jurisdictions) shall be provided in the site plans for surface plus subsurface BMPs. Local jurisdictions may develop a more stringent table (i.e., greater area required to be provided) at their discretion. In the absence of such a table, Table XI.1 shall be the default; and
- The site shall be configured such that runoff can be routed to BMPs located in the available area(s) of the site; and
- The site shall be laid out such that BMPs are located over infiltrative soils as practicable given the constraints of the site, unless infiltration is infeasible for risk-based reasons identified in **TGD Section 2.4**, and

- Satisfaction of these criteria shall be documented in exhibits or narrative descriptions.

OR

- A site specific study shall be prepared as part of the Project WQMP that documents that the site cannot be designed to allow more area for BMPs. The study may consider:
    - Site conditions/constraints (e.g., depth to groundwater, topography, existing utilities)
    - Zoning/code requirements (e.g., target density, accessibility, traffic circulation, health and safety, setbacks, etc.)
    - Economic feasibility

Table XI.1 provides the recommended percentage of a project site that is required to be made available for LID BMPs in order to meet minimum criteria for site design to allow BMPs.

Table XI.1: Recommended Minimum Criteria for Site Design

Project Type		Recommended effective area[1] required to be made available for LID BMPs (surface + subsurface facilities) to meet site design criteria[2] (percent of site)
New Development	SF/MF Residential < 7 du/ac	10
	SF/MF Residential 7 – 18 du/ac	7
	SF/MF Residential > 18 du/ac	5
	Mixed Use, Commercial, Institutional/Industrial w/ FAR < 1.0	10
	Mixed Use, Commercial, Institutional/Industrial w/ FAR 1.0 – 2.0	7
	Mixed Use, Commercial, Institutional/Industrial w/ FAR > 2.0	5
	Podium (parking under > 75% of project)	3
	Projects with zoning allowing development to lot lines	2
	Transit Oriented Development[3]	5
	Parking	5

Table XI.1: Recommended Minimum Criteria for Site Design

Project Type		Recommended effective area[1] required to be made available for LID BMPs (surface + subsurface facilities) to meet site design criteria[2] (percent of site)
Redevelopment	SF/MF Residential < 7 du/ac	5
	SF/MF Residential 7 – 18 du/ac	4
	SF/MF Residential > 18 du/ac	3
	Mixed Use, Commercial, Institutional/Industrial w/ FAR < 1.0	5
	Mixed Use, Commercial, Institutional/Industrial w/ FAR 1.0 – 2.0	4
	Mixed Use, Commercial, Institutional/Industrial w/ FAR > 2.0	3
	Podium (parking under > 75% of project)	2
	Projects with zoning allowing development to lot lines	1
	Transit Oriented Development[3]	3
	Projects in Historic Districts	3

[1] "Effective area" is defined as area which 1) is suitable for a BMP (for example, if infiltration is potentially feasible for the site based on infeasibility criteria, infiltration must be allowed over this area) and 2) receives runoff from impervious areas.
[2]Criteria for site design are only required to be met if the Project WQMP seeks to demonstrate that the full stormwater design volume cannot be feasibly managed on-site.
[3] Transit oriented development is defined as a development with development center within one half mile of a mass transit center.
Key: du/ac = dwelling units per acre, FAR = Floor Area Ratio = ratio of gross floor area of building to gross lot area
MF = Multi Family, SF = Single Family

The table is intended to be used in the feasibility process as follows:

- If a project seeks to demonstrate that it is not feasible to manage the entire design stormwater volume on-site, it is necessary to demonstrate that minimum criteria for site design have been met as part of making this determination by comparing the effective area provided for LID BMPs within the drainage are to the values in Table XI.1.
- If the percentage of the site recommended in Table XI.1 is provided and LID BMPs still does not achieve the stormwater design volume, then this allows for remaining volume to be met through alternative compliance. If the percentage of the site Table XI.1 is not provided for LID BMPs and the stormwater design volume is not managed, this provides grounds for a reviewer to request that additional area be made available for BMPs in the site design until either the percentage of the site in Table XI.1 is provided or the entire stormwater design volume is managed.
- The project may provide more area for LID BMPs if desired.

Local jurisdictions may choose to develop analogous tables more stringent (i.e., higher areas required to be provided) than Table XI.1. Projects that employ LID BMPs to retain the full stormwater design volume (as documented by the Project WQMP) are not required to demonstrate that they meet criteria for site design.

XI.5.2. Criteria for Selecting Design Infiltration Rate for Feasibility Evaluation

Infiltration factor of safety shall be selected based on criteria contained in **Appendix VII.4**, and shall not be less than 2.0 under any condition. The designer may provide a higher factor of safety in the design of BMPs as warranted by project-specific factors described in **Appendix VII.4**. For the purpose of designing BMPs to achieve the maximum feasible retention plus biotreatment, the acceptable factor of safety should be minimized through a commitment to thorough site investigation, use of effective pretreatment controls, good construction practices, the commitment to restore the infiltration rates of soils that are damaged by prior uses or construction practices, and the commitment to effective maintenance practices. In most case, it is believed that a factor of safety of 2.0 is attainable with these commitments; however this does not remove the responsibility of the designer to apply a prudent factor of safety based on project-specific considerations.

XI.5.3. Criteria for Identifying All Possible Harvested Water Demands

The intent of this section is to provide criteria for identifying all possible demands for harvested water. The following criteria shall be met to demonstrate that all potential demands for harvested water have been considered:

- Potential demands for harvested water shall include all *consistent and reliable demands for non-potable water*, as defined below, that do not conflict with codes or ordinances in place at the time of Project WQMP submittal and do not conflict with prior water rights claims,
- *Consistent and reliable demands for non-potable water* shall include those demands identified in **Appendix IX** and any other non-potable demands meeting the general criteria of **Appendix IX**:
    - Irrigation water demand, as estimated via methods described in **Appendix IX** or an equivalent method as approved by the local jurisdiction.
    - Indoor toilet flushing demand, as estimated via methods described in **Appendix IX** or an equivalent method as approved by the local jurisdiction. Occupancy estimates shall be based on the lowest forecasted average annual occupancy beyond 2 years of completion.
    - Industrial process water demand, vehicle wash water, evaporative cooling water, and other non-potable uses based on the criteria for calculating harvested water demand contained in **Appendix IX**, for processes not anticipated to change in the foreseeable future. For building uses anticipated to change, a good faith estimate of the minimum typical wet season harvested water demand shall be used to evaluate the feasibility of harvest and use systems.

- Reclaimed water supply shall be evaluated on a project-specific basis and subtracted from harvested water demands; in the absence of project-specific conditions of approval, reclaimed water available to the project shall take priority over use of harvested stormwater and should reduce the demand for harvested water by the amount of reclaimed water available. The basis for this priority is provided in **Appendix X.2.8.**

APPENDIX XII. CONCEPTUAL BIOTREATMENT SELECTION, DESIGN, AND MAINTENANCE CRITERIA

The purpose of this Appendix is to provide conceptual-level guidance for selection, design, and maintenance of biotreatment BMPs. This Appendix is intended to be used as a concise reference for the biotreatment BMP design philosophy.

This Appendix is not intended to provide BMP-specific guidance or design-level specifications. BMP-specific guidance for the recognized suite of available biotreatment BMPs is provided in BMP Fact Sheets in **TGD Section 4**.

This Appendix is not intended to be use for specific criteria. Detailed and prescriptive guidance for sizing and designing biotreatment to achieve the maximum feasible infiltration and ET is provided in **Appendix XI**.

XII.1. Definition of Biotreatment BMPs

Biotreatment BMPs are a broad class of structural LID BMPs that treat stormwater using a suite of treatment mechanisms characteristic of biologically active systems. The design of biotreatment BMPs should strive to achieve the following goals, as applicable:

- Foremost, the BMP should be designed to provide the highest possible pollutant removal, with emphasis on removal of pollutants of concern.
- The BMP should be aesthetically pleasing.
- The BMP should provide multiple benefits such as aesthetic enjoyment, wildlife habitat, open space, and/or support recreational use (i.e. be an element of a trail system);
- The BMP should include educational signage for visitors if appropriate; that
- Ancillary elements (fencing, gates, and access roads) should serve to mitigate risks (i.e. drowning, vandalism) and minimize costs of maintenance.

Biotreatment BMPs provide a variety of treatment mechanisms to remove both suspended and dissolved pollutants in urban storm water runoff. All biotreatment BMPs include treatment mechanisms that employ soil microbes and plants. Biotreatment BMPs may be either flow-based (limited storage) or volume-based (storage a key design component) and are designed to treat and discharge urban stormwater runoff to a downstream conveyance system. Biotreatment BMPs can be designed to promote infiltration and ET even though they are treat-and-release BMPs. Systems not designed primarily to infiltrate or evapotranspire stormwater may still reduce the volume of stormwater via infiltration and ET. If necessary to mitigate risks to structures, human health, or other concerns, a biotreatment BMP may also be lined to prevent infiltration of urban storm water runoff into the underlying soils.

Operations and maintenance of biotreatment BMPs should emphasize preservation of hydraulic function and the promotion of robust biological processes. Biotreatment BMPs typically utilize "soft" infrastructure (e.g., vegetative slope stabilization as opposed to rip rap slope stabilization) and therefore require an adaptive approach to maintenance and performance enhancement, more typical of landscape maintenance than maintenance of hard infrastructure.

Note that while biotreatment BMPs may provide habitat value, plant growth may damage infrastructure elements in the facility such as fencing, curbs, etc. This hazard can be mitigated by incorporating root barriers or through regular maintenance.

The following sections provide principles that should govern the design, operation, and maintenance of biotreatment BMPs installed to meet permit requirements in Orange County.

XII.2. Biotreatment Selection to Address Pollutants of Concern

Biotreatment BMPs shall be selected that provide unit operations and processes (UOPs) that address the project pollutants of concern. The process of biotreatment BMP selection shall consist of the following steps described in TGD Section 2.4.

XII.3. Conceptual Biotreatment Design Requirements

Biotreatment design requirements shall be consistent with the following principles:

- **Biotreatment BMPs shall be sized according to permit requirements described in the Section 2.4 of the Model WQMP.**
- **Biotreatment BMPs shall incorporate unit processes to address pollutants of concern. See TGD Section 2.4 for guidance.**
- **Biotreatment BMPs shall be designed to achieve the maximum feasible infiltration and ET by adhering to the criteria described in Appendix XI.**
- **Biotreatment BMPs shall be designed per the published design standards contained in the BMP Fact Sheets (Appendix XIV.5) and the design manuals referenced by these Fact Sheets.**
- **Biotreatment BMPs shall support a robust vegetative and microbial community appropriate to the local climate:**
  - For bioretention systems[18], select vegetation that is drought tolerant and can also survive extended periods of saturated soils.
  - For constructed stormwater wetlands and wet detention basins (wet ponds), select native species that include significant rhizomes and provide habitat benefits.

[18] The use of the term "bioretention systems" in this appendix refers to bioretention with underdrains, rain gardens with underdrains, planter boxes with underdrains, curb-extension planter boxes with underdrains, proprietary bioretention systems, and other similar BMPs.

- o For constructed stormwater wetlands and wet detention basins (wet ponds) provide appropriate mix of open water to vegetated area. The appropriate mix depends on the primary target constituents. For example, where nitrate is the dominant nutrient, the appropriate mix would include a higher proportion of vegetated area such as 80% vegetated, 20% open water.
- o For dry extended vegetated detention basins, vegetated swales, and filter strips, select a variety of plant species that are drought tolerant, but can also survive periodic inundation.
- o Provide an irrigation system, if necessary, for plant establishment and maintenance.

- **Biotreatment BMPs shall incorporate amended media and soils designed for the intended function of the BMP.**
  - o Select amended media for use in bioretention systems that is effective at removing pollutants of concern, can absorb and evapotranspirate runoff, and where appropriate, can facilitate infiltration.
  - o Select media and soils that will not potentially leach pollutants, specifically dissolved nutrients and metals in some cases.
  - o Amend soils in dry extended detention basins, swales, and filter strips to provide suitable soils for supporting plants, which can absorb and evapotranspire runoff and where appropriate facilitate infiltration.
  - o Design wet detention basins (wet ponds) and constructed stormwater wetlands using soils that support growth of attached plants.
- **BMPs hydraulics shall be designed to maximize pollutant removal functions.**
  - o For all biotreatment BMPs, design inlets or overland flow entry to BMPs to prevent scour or re-entrainment of pollutants.
  - o Provide maximum flow path distance between outlet and inlet and with sufficient length to width ratio to limit short circuiting.
  - o For constructed stormwater wetlands and wet detention basins, provide the storage capacity for the DCV in the wet pool at a minimum.
  - o Seasonal constructed stormwater wetlands and seasonal wet detention basins should not be used unless there is a reasonable expectation that tributary land uses will provide dry weather flows during seasonally wet period to maintain vegetation and prevent stagnant water.
  - o For constructed stormwater wetlands and wet detention basins designed to be continually wet (opportunities may be limited in Orange County), ensure that a low-flow source of water is present to maintain vegetation and prevent stagnant conditions.
  - o Design features shall allow for monitoring of drawdown such as depth markers and monitoring ports.
  - o For bioretention systems, provide media contact time sufficient for pollutant removal, with upper limitations on contact time to avoid leaching of retained pollutants. Traditional media should generally be designed in the range of 2 to 12 inches per hour, while specialized media can be effective for many pollutants of concern at much higher flowrates (residence times on the order of several minutes). For bioretention systems, design media mix and layer separation systems (i.e. between media and gravel layers) to reduce potential for clogging.

- For bioretention systems that include infiltration as a component, design a gravel pool below the underdrains (where used; ensure that the soils below this area can infiltrate (i.e., do not compact, or if compacted, restore soil infiltration capacity)). The minimum depth of gravel pool should be determined based on the underlying infiltration based on the amount of water that will infiltrate in 48 hours (see **Appendix XI.2**)
- For bioretention systems that will include infiltration as a component, the soil below the gravel pool must be able to allow infiltration. The soil may not be compacted. If the soil is compacted, the soil infiltration capacity must be restored.
- Consider using hydraulic control on the outlet of bioretention systems whenever practical rather than using media with lower infiltration rates for hydraulic control. This practice aids in avoiding clogging and can improve uniformity of performance over the life of the facility.
- For bioretention systems, do not use geotextile fabrics between layers of media due to clogging issues; use progressively-graded aggregate layers to prevent migration of fines if necessary.
- For bioretention systems limit ponding depths to 12 inches, unless system is isolated from public access via fencing or equivalent, then ponding depths should be limited to 18 inches.
- Bioretention systems and dry extended detention basins shall be designed to limit surface ponding to less than 96 hours for vector control per **California Department of Health Guidelines**. To provide a margin of safety, bioretention systems and extended detention basins should be designed to limit surface ponding to 72 hours. Subsurface ponding (in stone or gravel trenches) can create a vector hazard if the media has pore spaces that vectors can breed in.
- For biotreatment BMPs that employ extended detention, design outlet structures to ensure appropriate drawdown times and patterns and prevent floatables from leaving the facility; ensure that small storms receive appropriate extended detention times. A common rule of thumb is that the bottom half of the facility volume should draw down in two thirds of the total drawdown time.
- Outlet structures should be located and designed so that they are accessible for inspection and maintenance.
- For vegetated swales and filter strips, provide level spreaders and check dams where appropriate to promote even distribution of flow across the system.
- Design systems such that flows above the BMP design intensity are provided a flow route that bypasses the BMP or can be passed through the BMP without entraining soils, media, or captured pollutants.

- **Biotreatment BMPs shall be subject to rigorous construction oversight, acceptance, and documentation process.**
  - Provide construction oversight by trained professionals to ensure that the BMP is installed as designed.
  - Consider conducting a flow test for bioretention systems to ensure they function at the design level.
  - Require the preparation of as-built drawings that clearly indicated design features of the BMP and inlet and outlet systems.
  - Inspect BMPs after initial commissioning to ensure that they are functioning as intended. More frequent inspection during initial operation periods (i.e., first

rainy season) can help to mitigate early problems and ensure design level performance.

XII.4. Conceptual Biotreatment Operation Requirements

An operation and maintenance plan shall be developed for biotreatment BMPs that includes the following elements:

- **Frequency and type of inspections,**
- **Observations during wet weather to visually observe whether the BMP is functioning as intended,**
- **List of parameters/checklists for identifying maintenance needs and triggering maintenance activities,**
- **Vegetation management plan, including routine maintenance, and irrigation, if necessary,**
- **Sediment, trash and debris removal, and**
- **Routine and major (infrequent) maintenance activities.**

Reclaimed water considerations for operation of biotreatment BMPs:

If the project utilizes reclaimed water for irrigation, the project is required to comply with all waste discharge requirements and water provider use requirements applicable to the project. It is the responsibility of the project owner to ensure that operation of the project complies with these requirements. It is the responsibility of the water provider to ensure that requirements associated with the use of reclaimed water result in BMP operations that are protective of receiving water quality.

XII.5. Conceptual Biotreatment Maintenance Requirements

Biotreatment maintenance requirements contained in the Project O&M Plan shall be consistent with the following principles:

- **Routine maintenance shall be provided to ensure consistently high performance and extend facility life.**
  - Maintain vegetation and media to perpetuate a robust vegetative and microbial community (thin/trim vegetation, replace spent media and mulch).
  - Periodically remove dead vegetative biomass to prevent export of nutrients or clogging of the system.
  - Remove accumulated sediment before it significantly interferes with system function.
  - Where filtration/infiltration is employed, conduct maintenance to prevent surface clogging (surface scarring, raking, mulch replacement, etc.).
  - Add energy dissipation and scour-protection as required based on facility inspection.
  - Routinely remove accumulated sediment at the inlet and outlet and trash and debris from the entire BMP.
- **Major maintenance shall be provided when the performance of the facility declines significantly and cannot be restored through routine maintenance.**

- Replace media / planting soils as triggered by reduction in filtration/infiltration rates or decline in health of biological processes.
- Provide major sediment removal to restore volumetric capacity of basin-type BMPs.
- Repair or modify inlets/outlets to restore original function or enhance function based on observations of performance.

Detailed descriptions of BMP maintenance activities are provided in:

- Los Angeles County Stormwater BMP Operations and Maintenance Manual, Chapter 5: http://dpw.lacounty.gov/DES/design_manuals/StormwaterBMPDesignandMaintenance.pdf

APPENDIX XIII. THRESHOLD INCREMENTAL BENEFIT CRITERION

XIII.1. Intended Application

The following criterion is intended to be applied as part of determining the maximum feasible retention volume as part of the BMP selection and design process:

If a hypothetical BMP is designed to achieve the maximum feasible retention per the criteria contained **Appendix XI.3**, and, meeting these criteria, the BMP would achieve less than 40 percent capture of average annual runoff, then it is not mandatory to use the given BMP in order to demonstrate that the system has been designed to achieve the maximum feasible retention of the DCV.

This criterion does not suspend the requirements to (1) consider all applicable HSCs that are designed to provide retention, (2) conduct a rigorous feasibility analysis of all other retention BMPs before moving to biotreatment, and (3) to design biotreatment BMPs, if used, to achieve the maximum feasible infiltration and ET. As a result, the application of this criterion does not result in an "all or nothing" scenario for retention; rather it is intended to provide an objective basis for identifying BMPs for which *costs* (due to resulting multiple BMPs being required would) *greatly outweigh pollution control benefits*. In this case, the criterion allows the project to distribute the DCV to more cost-effective BMPs and still achieve retention with HSCs and biotreatment BMPs.

Based on the analysis described in **Appendix III.6**, a BMP designed for the full DCV will exceed 40 percent capture (and therefore be a mandatory consideration) if the storage can be recovered in 720 hours (30 days) or faster. Therefore this criterion would only apply in extremely limited cases where the DCV cannot be drained in less than 30 days. Generally, it will only apply to harvest and use systems where demand is extremely limited to manage the DCV.

This criterion does not apply to HSC (e.g., downspout disconnection, rain barrels), which are relatively inexpensive compared to engineered harvest and use systems and are commonly designed with the intent of providing relatively small incremental benefit to contribute to an overall effective system. HSCs must be considered wherever there are opportunities for their use.

XIII.2. Regulatory Basis

The Santa Ana Regional Water Quality Control Board MS4 Permit (**Order R8-2009-0030**) ("North County Permit") and the San Diego Regional Water Quality Control Board MS4 Permit (**Order R9-2009-0002**) ("South County Permit) have been adopted with specific requirements for new development and significant redevelopment stormwater control. Both permits are based on the MEP[19] standard included in the 1987 amendments of the Clean Water Act.

The permits require "retention" (meaning no surface or piped discharges) of stormwater on site as the first alternative, LID BMPs, and allow biotreatment BMPs to be considered only after infiltration, harvest and use, and ET cannot be feasibly implemented to address the entire DCV. The South County Permit requires a *"technical feasibility analysis including cost benefit analysis"* (F.1.d(7)(b)). The North County Permit, by way of its description of the MEP standard (see Footnote 19), requires the consideration of multiple interrelated factors in assessing feasibility. The North Orange County Permit also allows waivers of BMP requirements to be granted *"...if the cost of BMP implementation greatly outweighs the pollution control benefits..."* (XII.E.1). Therefore, there is sound regulatory basis for the consideration of cost-effectiveness, societal factors, and effects on other media, in addition to physical/technical factors, in the evaluation of feasibility of retention on-site.

For example, it would nearly always be physically feasible to install a tank to store the DCV for a project for subsequent use of captured water. However, unless sufficient demand for the captured water exists to empty the tank relatively quickly between storm events, the tank would be relatively ineffective for stormwater management. If the tank was on-line, then it would in effect behave primarily as a wet-vault, whose performance is typically much less than biotreatment. If it was off-line (tank is bypassed when full), then there would be significant untreated flows.

While a system with a low demand would technically fulfill the volumetric LID performance criteria contained in the permits (South County Permit at F.1.d(4)(d)(i), and North County Permit at XII.C.2), this system would be inconsistent with the intent of the permits, and would not meet the MEP requirement and therefore should not be encouraged or mandated. The cost and potential effects on other media associated with such a system would greatly outweigh the pollution control benefits it provides. The other environmental and societal effects associated with such a system include:

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[19] The North County Permit describes MEP as follows: "MEP is not defined in the Clean Water Act; it refers to management practices, control techniques, and system, design and engineering methods for the control of pollutants taking into account considerations of synergistic, additive, and competing factors, including, but not limited to, gravity of the problem, technical feasibility, fiscal feasibility, public health risks, societal concerns, and social benefits."

- Energy and resources used to manufacture of plastic, metal, or concrete tanks,
- Energy and resources used manufacture of pumps, treatment systems, and piping,
- Energy and air quality impacts associated with shipping and installing the system
- Energy and air quality impacts associated with transportation for specialized maintenance activities
- Disposal of system elements at the end of usable life.

XIII.3. Comparison to Anticipated Performance of Alternative Scenario

The numeric threshold should reflect conditions where *the cost of BMP implementation greatly outweighs the pollution control benefits* and where the *"alternative scenario"* allowed by the criterion provides similar effectiveness and much lower cost. For both infiltration BMPs and harvest and use BMPs, this can be referenced to the volume reduction and treatment performance that would be achieved by biotreatment BMPs designed for the maximum feasible partial retention (i.e., the *alternative scenario*).

In the case that infiltration and harvest and use are not feasible, the alternative scenario is biotreatment BMPs designed for the maximum partial retention. Biotreatment BMPs must be designed to achieve the maximum feasible retention and ET of stormwater per the specific criteria contained in **Appendix XI**, and must be designed to biotreat runoff as feasible up to 80 percent average annual capture efficiency.

When designed to these criteria, biotreatment BMPs are expected to achieve retention of a substantial volume of stormwater. A recent analysis of the monitored inflow and outflow data contained in the International Stormwater BMP Database showed a volume reduction on the order of 40 percent for biofilters, 30 percent for extended detention basins, and 60 percent for bioretention areas.

Table XIII.1: Volume Reduction Summary of Biotreatment BMP Categories in the International Stormwater BMP Database

BMP Category	# of Monitoring Studies	25th Percentile	Median	75th Percentile	Average
Biofilter – Grass Strips	16	18%	34%	54%	38%
Biofilter – Grass Swales	13	35%	42%	65%	48%
Bioretention (with underdrains)	7	45%	57%	74%	61%
Detention Basins – Surface, Grass Lined	11	26%	33%	43%	33%

NOTES

Relative volume reduction = (Study Total Inflow Volume - Study Total Outflow Volume)/(Study Total Inflow Volume)

Excluded other categories due to lack of sufficiently robust dataset or inability to conduct reasonableness screening.

Summary does not reflect performance categorized according to storm size (bin).

Source: International Stormwater Best Management Practices (BMP) Database, Technical Summary: Volume Reduction. January 2011. http://www.bmpdatabase.org/Docs/Volume%20Reduction%20Technical%20Summary%20Jan%202011.pdf

These values provide a benchmark reference for establishing an incremental threshold criterion. Retention BMPs should provide significantly greater volume reduction than the volume reduction achieved by biotreatment BMPs. Otherwise, there is no basis for requiring retention BMPs when biotreatment BMPs would provide equivalent volume reduction and provide treatment of captured water that is not retained, thereby not requiring a separate BMP to be added (at additional cost) to meet the remaining biotreatment obligations. On this basis, a threshold incremental benefit of approximately 40 percent is appropriate.

APPENDIX XIV. BMP FACT SHEETS

This appendix contains BMP fact sheets for the following BMP categories:

Hydrologic Source Control Fact Sheets (HSC)
- HSC-1: Localized On-Lot Infiltration
- HSC-2: Impervious Area Dispersion
- HSC-3: Street Trees
- HSC-4: Residential Rain Barrels
- HSC-5: Green Roof / Brown Roof
- HSC-6: Blue Roof

Infiltration BMP Fact Sheets (INF)
- INF-1: Infiltration Basin Fact Sheet
- INF-2: Infiltration Trench Fact Sheet
- INF-3: Bioretention with no Underdrain
- INF-4: Bioinfiltration Fact Sheet
- INF-5: Drywell
- INF-6: Permeable Pavement (concrete, asphalt, and pavers)
- INF-7: Underground Infiltration

Harvest and Use BMP Fact Sheets (HU)
- **HU-1: Above-Ground Cisterns**
- **HU-2: Underground Detention**

Biotreatment BMP Fact Sheets (BIO)
- **BIO-1: Bioretention with Underdrains**
- **BIO-2: Vegetated Swale**
- **BIO-3: Vegetated Filter Strip**
- **BIO-4: Wet Detention Basin**
- **BIO-5: Constructed Wetland**
- BIO-6: Dry Extended Detention Basin
- **BIO-7: Proprietary Biotreatment**

Treatment Control BMP Fact Sheets (TRT)
- TRT-1: Sand Filters
- TRT-2: Cartridge Media Filter

Note: ET plays an important role in the performance of HSC, INF, HU, and BIO BMPs. However, specific fact sheets for ET are not included. Criteria for designing BMPs to achieve the maximum feasible infiltration and ET are contained in **Appendix XI**.

The BMP designs described in these fact sheets and in the referenced design manuals shall constitute what are intended as LID and Treatment Control BMPs for the purpose of meeting stormwater management requirements. Other BMP types and variations on these designs may be approved at the discretion of the reviewing agency if documentation is provided demonstrating similar functions and equivalent or better expected performance.

XIV.1. Hydrologic Source Control Fact Sheets (HSC)

HSC-1: Localized On-Lot Infiltration

'Localized on-lot infiltration' refers to the practice of collecting on-site runoff from small distributed areas within a catchment and diverting it to a dedicated on-site infiltration area. This technique can include disconnecting downspouts and draining sidewalks and patios into french drains, trenches, small rain gardens, or other surface depressions. For downspout disconnections and other impervious area disconnection involving dispersion over pervious surfaces, but without intentional ponding, see HSC-2: Impervious Area Dispersion.

Also known as:

- *Downspout infiltration*
- *Retention grading*
- *French drains*
- *On-lot rain gardens*

On-lot rain garden
Source: lowimpactdevelopment.org

Feasibility Screening Considerations

- 'Localized on-lot infiltration' shall meet infiltration infeasibility screening criteria to be considered for use.

Opportunity Criteria

- Runoff can be directed to and temporarily pond in pervious area depressions, rock trenches, or similar.
- Soils are adequate for infiltration or can be amended to provide an adequate infiltration rate.
- Shallow utilities are not present below infiltration areas.

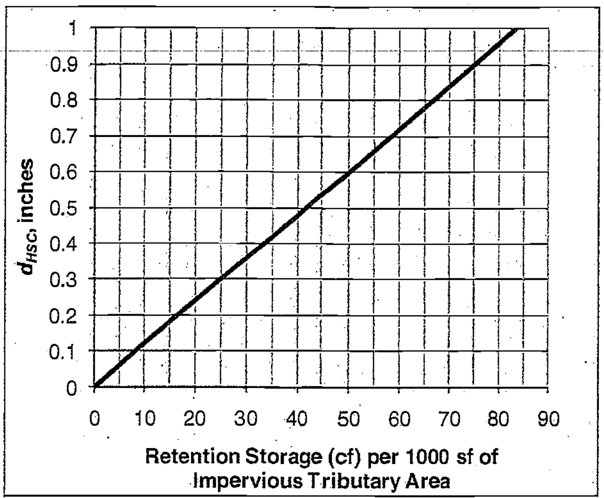
OC-Specific Design Criteria and Considerations

- ☐ A single on-lot infiltration area should not be sized to retain runoff from impervious areas greater than 4,000 sq. ft.; if the drainage area exceeds this criteria, sizing should be based on calculations for bioretention areas or infiltration trenches.
- ☐ Soils should be sufficiently permeable to eliminate ponded water within 24 hours following a 85th percentile, 24-hour storm event.
- ☐ Maximum ponding depth should be should be less than 3 inches and trench depth should be less than 1.5 feet.
- ☐ Infiltration should not be used when the depth to the mounded seasonally high table is within 5 feet of the bottom of infiltrating surface.
- ☐ Infiltration via depression storage, french drains, or rain gardens should be located greater than 8 feet from building foundations.
- ☐ Site slope should be less than 10%.
- ☐ Infiltration unit should not be located within 50 feet of slopes greater than 15 percent.
- ☐ Side slopes of rain garden or depression storage should not exceed 3H:1V.
- ☐ Effective energy dissipation and uniform flow spreading methods should be employed to prevent erosion resulting fromwater entering infiltration areas.

☐ Overflow should be located such that it does not cause erosion orand is conveyed away from structures toward the downstream conveyance and treatment system. .

Calculating HSC Retention Volume

- The retention volume provided by localized on-lot infiltration can be computed as the storage volume provided by surface ponding and the pore space within an amended soil layer or gravel trench.
- Estimate the average retention volume per 1000 square feet impervious tributary area provided by on-lot infiltration.
- Look up the storm retention depth, d_{HSC} from the chart to the right.
- The max d_{HSC} is equal to the design capture storm depth for the project site.



Configuration for Use in a Treatment Train

- Localized on-lot infiltration would typically serve as the first in a treatment train and should only be used where tributary areas do not generate significant sediment that would require pretreatment to mitigate clogging.
- The use of impervious area disconnection reduces the sizing requirement for downstream LID and/or conventional treatment control BMPs.

Additional References for Design Guidance

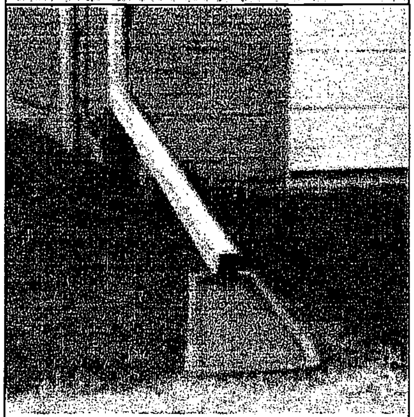
- LID Center – Rain Garden Design Template. http://www.lowimpactdevelopment.org/raingarden_design/
- University of Wisconsin Extension. Rain Gardens: A How-To Manual for Homeowners. http://learningstore.uwex.edu/assets/pdfs/GWQ037.pdf

HSC-2: Impervious Area Dispersion

Impervious area dispersion refers to the practice of routing runoff from impervious areas, such as rooftops, walkways, and patios onto the surface of adjacent pervious areas. Runoff is dispersed uniformly via splash block or dispersion trench and soaks into the ground as it move slowly across the surface of pervious areas. Minor ponding may occur, but it is not the intent of this practice to actively promote localized on-lot storage (See HSC-1: Localized On-Lot Infiltration).

Also known as:

- *Downspout disconnection*
- *Impervious area disconnection*
- *Sheet flow dispersion*



Simple Downspout Dispersion
Source:
toronto.ca/environment/water.htm

Feasibility Screening Considerations

- Impervious area dispersion can be used where infiltration would otherwise be infeasible, however dispersion depth over landscaped areas should be limited by site-specific conditions to prevent standing water or geotechnical issues.

Opportunity Criteria

- Rooftops and other low traffic impervious surface present in drainage area.
- Soils are adequate for infiltration. If not, soils can be amended to improve capacity to absorb dispersed water (see MISC-2: Amended Soils).
- Significant pervious area present in drainage area with shallow slope
- Overflow from pervious area can be safely managed.

OC-Specific Design Criteria and Considerations

- [ ] Soils should be preserved from their natural condition or restored via soil amendments to meet minimum criteria described in Section .
- [ ] A minimum of 1 part pervious area capable of receiving flow should be provided for every 2 parts of impervious area disconnected.
- [ ] The pervious area receiving flow should have a slope ≤ 2 percent and path lengths of ≥ 20 feet per 1000 sf of impervious area.
- [ ] Dispersion areas should be maintained to remove trash and debris, loose vegetation, and protect any areas of bare soil from erosion.
- [ ] Velocity of dispersed flow should not be greater than 0.5 ft per second to avoid scour.

Calculating HSC Retention Volume

- The retention volume provided by downspout dispersion is a function of the ratio of impervious to pervious area and the condition of soils in the pervious area.
- Determine flow patterns in pervious area and estimate footprint of pervious area receiving dispersed flow. Calculate the ratio of pervious to impervious area.
- Check soil conditions using the soil condition design criteria below; amend if necessary.
- Look up the storm retention depth, d_{HSC} from the chart below.

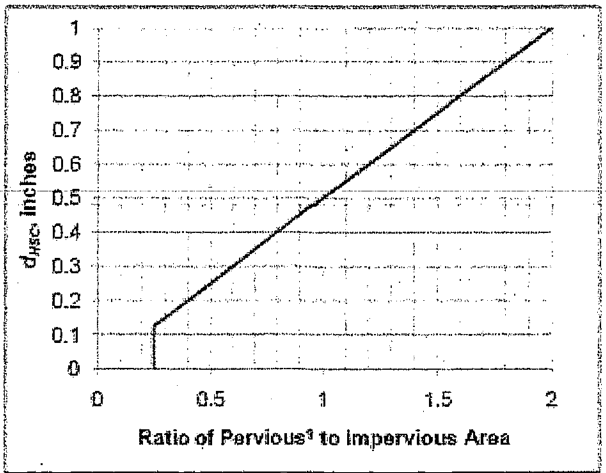
- The max d_{HSC} is equal to the design storm depth for the project site.

Soil Condition Design Criteria

- ☐ Maximum slope of 2 percent
- ☐ Well-established lawn or landscaping
- ☐ Minimum soil amendments per criteria in MISC-2: Amended Soils.

Configuration for Use in a Treatment Train

- Impervious area disconnection is an HSC that may be used as the first element in any treatment train
- The use of impervious area disconnection reduces the sizing requirement for downstream LID and/or treatment control BMPs



[1] Pervious area used in calculation should only include the pervious area receiving flow, not pervious area receiving only direct rainfall or upslope pervious drainage.

Additional References for Design Guidance

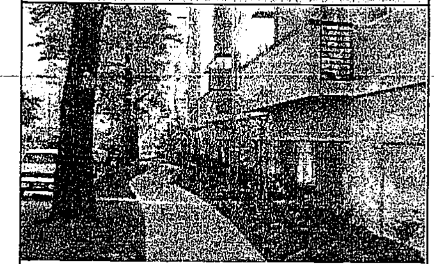
- SMC LID Manual (pp 131) http://www.lowimpactdevelopment.org/guest75/pub/All_Projects/SoCal_LID_Manual/SoCalLID_Manual_FINAL_040910.pdf
- City of Portland Bureau of Environmental Services. 2010. How to manage stormwater – Disconnect Downspouts. http://www.portlandonline.com/bes/index.cfm?c=43081&a=177702
- Seattle Public Utility: http://www.cityofseattle.org/util/stellent/groups/public/@spu/@usm/documents/webcontent/spu01_006395.pdf
- Thurston County, Washington State (pp 10): http://www.co.thurston.wa.us/stormwater/manual/docs-faqs/DG-5-Roof-Runoff-Control_Rev11Jan24.pdf

HSC-3: Street Trees

By intercepting rainfall, trees can provide several aesthetic and stormwater benefits including peak flow control, increased infiltration and ET, and runoff temperature reduction. The volume of precipitation intercepted by the canopy reduces the treatment volume required for downstream treatment BMPs. Shading reduces the heat island effect as well as the temperature of adjacent impervious surfaces, over which stormwater flows, and thus reduces the heat transferred to downstream receiving waters. Tree roots also strengthen the soil structure and provide infiltrative pathways, simultaneously reducing erosion potential and enhancing infiltration.

Also known as:

➢ *Canopy interception*



Street trees
Source: Geosyntec Consultants

Feasibility Screening Considerations

- Not applicable

Opportunity Criteria

- Street trees can be incorporated in green streets designs along sidewalks, streets, parking lots, or driveways.
- Street trees can be used in combination with bioretention systems along medians or in traffic calming bays.
- There must be sufficient space available to accommodate both the tree canopy and root system.

OC-Specific Design Criteria and Considerations

☐ Mature tree canopy, height, and root system should not interfere with subsurface utilities, suspended powerlines, buildings and foundations, or other existing or planned structures. Required setbacks should be adhered to.

☐ Depending on space constarints, a 20 to 30 foot diameter canopy (at maturity) is recommended for stormwater mitigation.

☐ Native, drought-tolerant species should be selected in order to minimize irrigation requirements and improve the long-term viability of trees.

☐ Trees should not impede pedstrian or vehicle sight lines.

☐ Planting locations should receive adequate sunlight and wind protection; other environmental factors should be considered prior to planting.

☐ Frequency and degree of vegetation management and maintenance should be considered with respect to owner capabilities (e.g., staffing, funding, etc.).

☐ Soils should be preserved in their natural condition (if appropriate for planting) or restored via soil amendments to meet minimum criteria described in MISC-2: Amended Soils. If necessary, a landscape architect or plant biologist should be consulted.

☐ A street tree selection guide, such as that specific to the City of Los Angeles, may need to be consulted to select species appropriate for the site design constraints (e.g., parkway size, tree height, canopy spread, etc.)

☐ Infiltration should not cause geotechnical hazards related to adjacent structures (buildings, roadways, sidewalks, utilities, etc.)

Calculating HSC Retention Volume

- The retention volume provided by streets trees via canopy interception is dependent on the tree species, time of the year, and maturity.
- To compute the retention depth, the expected impervious area covered by the full tree canopy after 4 years of growth must be computed (IA_{HSC}). The maximum retention depth credit for canopy interception (d_{HSC}) is 0.05 inches over the area covered by the canopy at 4 years of growth.

Configuration for Use in a Treatment Train

- As a HSC, street trees would serve as the first step in a treatment train by reducing the treatment volume and flow rate of a downstream treatment BMP.

Additional References for Design Guidance

- California Stormwater BMP Handbook. http://www.cabmphandbooks.com/Documents/Development/Section_3.pdf
- City of Los Angeles, Street Tree Division - Street Tree Selection Guide. http://bss.lacity.org/UrbanForestryDivision/StreetTreeSelectionGuide.htm
- Portland Stormwater Management Manual. http://www.portlandonline.com/bes/index.cfm?c=35122&a=55791
- San Diego County – Low Impact Development Fact Sheets. http://www.sdcounty.ca.gov/dplu/docs/LID-Appendices.pdf

HSC-4: Residential Rain Barrels

Rain barrels are above ground storage vessels that capture runoff from roof downspouts during rain events and detain that runoff for later reuse for irrigating landscaped areas. The temporary storage of roof runoff reduces the runoff volume from a property and may reduce the peak runoff velocity for small, frequently occurring storms. In addition, by reducing the amount of storm water runoff that flows overland into a storm water conveyance system (storm drain inlets and drain pipes), less pollutants are transported through the conveyance system into local creeks and ocean. The reuse of the detained water for irrigation purposes leads to the conservation of potable water and the recharge of groundwater.

Also known as:

- Small cistern

Rain Barrel
Source: http://www.auburn.edu/projects/sustainability/website/newsletter/0910.php

Feasibility Screening Considerations

- Rain barrels not actively managed that overflow to infiltration areas shall be screened as Infiltration BMPs for feasibility screening.

Opportunity Criteria

- Rooftops with downspouts or other suitable conveyances (e.g. rain chains) present in the drainage area.
- If detained water will be used for irrigation, sufficient vegetated areas and other impervious surfaces must be present in drainage area.
- Storage capacity and sufficient area for overflow dispersion must be accounted for.

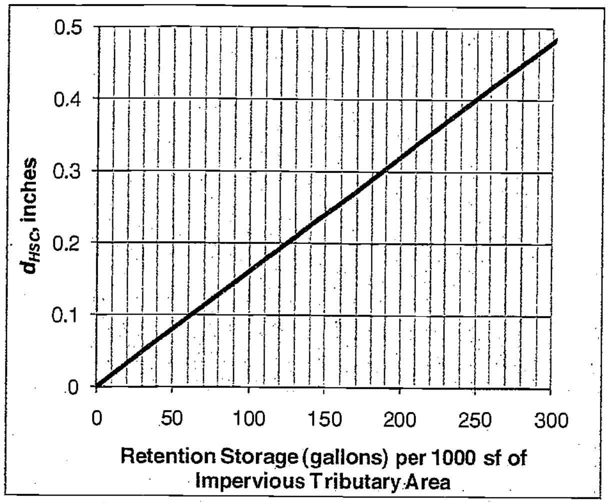
OC-Specific Design Criteria and Considerations

- ☐ Screens on gutters and downspouts should be used to remove sediment and particles as the water enters the barrel or cistern. Removable child-resistant covers and mosquito screening should be used to prevent unwanted access.
- ☐ Above-ground barrels should be secured in place.
- ☐ Above-ground barrels should not be located on uneven or sloped surfaces; if installed on a sloped surface, the base where the cistern will be installed should be leveled prior to installation.
- ☐ Overflow dispersion should occur greater than 8 feet from building foundations.
- ☐ Dispersion should not cause geotechnical hazards related to slope stability.
- ☐ Dispersion should be only allowed to stable vegetated areas where erosion or suspension of sediment is minimized.
- ☐ Effective energy dissipation and uniform flow spreading methods should be employed to prevent erosion and facilitate dispersion.
- ☐ Aesthetics should be considered for placement of barrels and incorporation into surroundings. Placement should allow easy access for regular maintenance.

☐ To draw down a 55 gallon rain barrel within 2 days with plant watering, at least 1,600 square feet of conservation landscape or 800 square feet of active turf area is needed.

Calculating HSC Retention Volume

- At least 1,600 sq-ft of conservation landscape or 800 sq-ft of active turf landscape shall be provided for each rain barrel to claim an HSC credit volume
- The effective volume provided by rain barrels that are not actively managed can be computed as 50% of the total storage volume (e.g., 27.5 gallons for each 55 gallon barrel.
- If the rain barrel is actively managed then it should be treated as a cistern as described in **Appendix XIV.4**.
- Estimate the average retention volume per 1000 square feet impervious tributary area provided by rain barrels. Example:
  - 500 square feet of roof draining to a 55 gallon rain barrel
  - Retention volume = (55/2) = 27.5 gallons
  - Retention volume per 1000 sq feet = 27.5 gallons/ 0.5 = 55 gallons per 1000 sq-ft
  - Based on the retention storage estimated, look up the storm retention depth, d_{HSC} from the chart to the right = 0.07 inches
  - The max d_{HSC} is equal to the design storm depth for the project site.



Configuration for Use in a Treatment Train

- Rain barrels can be combined into a treatment train to provide enhanced water quality treatment and reductions in the runoff volume and rate. For example, if a green roof is placed upgradient of a rain barrel, the rate and volume of water flowing to the barrel can be reduced and the water quality enhanced.
- Rain barrels can be incorporated into the landscape design of a site and can be aesthetically pleasing as well as functional for irrigation purposes.

Additional References for Design Guidance

- Santa Barbara BMP Guidance Manual, Chapter 6: http://www.santabarbaraca.gov/NR/rdonlyres/91D1FA75-C185-491E-A882-49EE17789DF8/0/Manual_071008_Final.pdf
- County of Los Angeles Low Impact Development Standards Manual: http://dpw.lacounty.gov/wmd/LA_County_LID_Manual.pdf
- SMC LID Manual (pp 114): http://www.lowimpactdevelopment.org/guest75/pub/All_Projects/SoCal_LID_Manual/SoCalLID_Manual_FINAL_040910.pdf
- San Diego County LID Handbook Appendix 4 (Factsheet 26): http://www.sdcounty.ca.gov/dplu/docs/LID-Appendices.pdf