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A DESIGN MANUAL FOR SIZING INFILTRATION PONDS

by

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Stormwater infiltration facilities help reduce the hydrologic impacts of residential and commercial development. The design of these facilities is particularly challenging because of large uncertainties associated with predictions of both short-term and long-term infiltration rates. This manual describes step-by-step procedures for collecting and analyzing data and information needed to size infiltration ponds. The procedures were developed recognizing that the performance of infiltration facilities depends upon a combination of near-surface soil characteristics, subsurface geology, groundwater conditions, and pond geometry. The manual focuses on infiltration ponds located in unconsolidated geologic materials.

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1 INTRODUCTION

Storm water infiltration facilities are used to reduce the hydrologic impacts of residential and commercial development. Increased runoff caused by impervious surfaces within these developments may destabilize stream channels and may degrade or destroy fish and wildlife habitat. Impervious surfaces also prevent rain and snowmelt from seeping into the ground and recharging streams, wetlands, and aquifers. Infiltration facilities, such as ponds, dry wells, infiltration galleries, and swales, are designed to capture and retain runoff and allow it to infiltrate rather than to discharge directly to surface water. Important benefits of groundwater infiltration facilities include reducing surface-runoff volume, reducing pollutant discharge, reducing thermal impacts on fisheries, increasing groundwater recharge, and augmenting low-flow stream conditions.

The design of infiltration facilities is particularly challenging because of the large uncertainties associated with predictions of both short-term and long-term infiltration rates. These uncertainties in infiltration rates translate into uncertainties in the area and volume that are required for infiltration ponds. There are economic penalties for both under-designed and over-designed facilities. Under-sized ponds may result in flooding, while over-sized ponds may be inefficient in terms of land use and expensive in terms of property acquisition.

This manual describes step-by-step procedures for collecting and analyzing data and information needed to size infiltration ponds. The procedures were developed recognizing that the performance of infiltration facilities depends upon a combination of near-surface soil characteristics, subsurface geology, groundwater conditions, and pond geometry. The manual focuses on infiltration ponds located in unconsolidated geologic materials. The performance of ponds or other infiltration facilities in geologic environments comprised of fractured rock are not considered. The manual does not directly consider water quality issues in terms of fate and transport of constituents within the stormwater runoff. It focuses on estimating the rate at which this stormwater will infiltrate into the subsurface, and not on chemical and biological processes that may affect groundwater water quality.

The approaches described below are aimed at developing design recommendations and at identifying situations in which more sophisticated approaches are likely warranted or recommended. The approaches are intended to provide complementary or equivalent alternatives to the approaches described in the WDOE manual (2001).

1.1 Performance Objectives

This manual considers analytical techniques and approaches for sizing infiltration ponds to meet the performance objectives specified in Volumes III and V of the Washington Department of Ecology's Stormwater Management Manual (WDOE, 2001). The performance objectives for infiltration facilities described in the WDOE manual are based on two primary considerations: a water quality consideration related to how quickly the pond empties after a storm event and a water quantity consideration related to the magnitude of overflow from the facility if the facility includes overflow features or design components (WDOE, 2001, Vol. III, page 3-73). For the water quality consideration related to how quickly the pond empties, the infiltration facility must be designed to drain completely within 24 hours after the flow to it has stopped. This requirement is aimed at preventing anaerobic (low-oxygen) conditions from developing in and beneath the infiltration facility. According to the WDOE manual (WDOE, 2001, Vol. III, page 3-73), these anaerobic conditions can foster the growth of bacteria and may contribute to soil clogging and fouling. It should be noted that growth of bacteria and subsequent clogging and fouling can also be controlled through a well-designed and fully-implemented maintenance program.

The performance objective for water quantity considerations relates to overflow from infiltration facilities. Minimum Technical Requirement #7 (MTR #7) in the WDOE Stormwater Management Model (Volume I, Chapter 2, page 2-31) specifies that stormwater discharges to streams must mimic certain aspects of pre-developed conditions. Both the duration of stormwater flows and the peak discharge rates for a specified set of selected storms must be the same for post-development conditions as they were for pre-development conditions. In some cases, infiltration ponds are designed with an overflow that discharges to streams. In these cases, the rate of overflow will depend to some degree on infiltration rates. If infiltration rates are too low, the overflow rates may become large enough to cause violations of MTR #7.

The techniques and procedures described in this manual are not only applicable to infiltration ponds designed to meet the WDOE performance requirements. These techniques can be used to help size infiltration ponds for any project where a design objective is to infiltrate a prescribed amount of water in a prescribed amount of time. This type of performance objective is often used by state and federal agencies, as summarized in Table 1.

1.2 Manual Format

The format used in this manual is focused on a step-by-step methodology that is illustrated in Figure 1. The look-up tables and graphs are generally based on the results described in Massmann et al. (2003). The computer simulations used in Massmann et al. (2003) are loosely based on the Lacy Lid facility in Thurston County, Washington. This facility, which is described in Wiltsie (1998), was not chosen because it represents a particularly good or a particularly poor design example. Rather, it is typical of facilities in western Washington in terms of size, performance, and availability of data.

Methods for conducting full-scale infiltration tests and for evaluating the data from these tests are described in Appendix A. Appendix B includes example calculations for estimating infiltration rates using the design approach and equations described in this manual.

2 STEPS FOR SIZING INFILTRATION FACILITIES

The steps described below are aimed at achieving two main objectives. The first objective is to describe procedures used to develop designs for infiltration facilities. These designs will include recommendations for the size and shape of infiltration ponds or infiltration trenches. The second objective is to help identify situations in which more refined and sophisticated analyses should be brought to bear in the design process. These more sophisticated analyses may include additional field or laboratory tests and more realistic analytical tools including analytical groundwater flow models and computer simulations.

2.1 Estimate the volume of stormwater that must be infiltrated by the proposed or planned facility

The volume of stormwater that must be infiltrated and the rate at which this must occur are generally specified by local, regional, or state requirements. In many cases, the volume and required rates of discharge are controlled by both water quality and water quantity concerns, as introduced in Section 1. Although the methods for estimating the stormwater volume is beyond the scope of this manual, it is useful to recognize the general ways in which this stormwater volume or discharge can be described.

There are three primary ways in which the stormwater discharges are generally described: 1) a single-value or fixed volume of runoff water, 2) a single-event or single-storm runoff hydrograph (i.e. runoff volume versus time), and 3) a continuous hydrograph that considers multiple events or storms over some longer period of time. The first and most simple way of describing stormwater discharge is as a total volume of water that much be infiltrated in a prescribed period of time. This is the approach that is generally used for water quality considerations related to how quickly the pond empties. Table 1 describes time requirements used by different state and federal agencies. For example, the WDOE design requirement that the pond empty within 24 hours focuses on the runoff that is generated by a storm that is defined as the 24-hour storm with a 6-month return frequency (a.k.a., 6-month, 24-hour storm). (Note: For areas in western

Washington, the 6-month, 24-hour storm can be estimated as 72% of the 2-year, 24-hour rainfall amount.). The volume of run-off from this storm that is discharged to the infiltration facility is often determined using a “curve-number” method that relates a land area's total runoff volume to the precipitation it receives and to its natural storage capacity. This approach is described in Volume III, Chapter 2, Section 2.3.2 of the WDOE manual (2001). The important thing to note about this approach is that it results in a single number that represents the total volume of water that will be discharged to the stormwater facility, denoted as V_{design} in the sections below.

The second method for describing stormwater discharges is the single-event or single-storm runoff hydrograph. The hydrograph describes the volume of runoff versus time for some specified precipitation event. An example single-event hydrograph for a 6-month, 24 hour storm in Western Washington is included Figure 2. The single-event hydrograph method is used primarily for calculations aimed at insuring that the pond empties within 24 hours. For example, the WDOE stormwater manual (2001) describes the Santa Barbara Unit Hydrograph (SBUH) method for estimating the hydrograph from the 6-month, 24-hour storm. This description is included in Volume III, Chapter 2, Section 2.3.2 of the WDOE manual (2001). Because the unit hydrograph approach results in a time-series of inflows to the infiltration facility, spreadsheet solutions are generally required to account for the balances between time-dependent inflow, outflow, infiltration rates, and pond storage changes. These time-varying conditions generally preclude more simple designed approaches that are based on tables or graphs. (For example, Akan (2002) gives solutions for time-dependent infiltration rates that result from time-varying inflow rates and subsurface saturation conditions).

The third and most involved method for describing stormwater discharges uses a continuous hydrograph that considers the runoff from multiple events or storms over some longer period of time. This approach generally requires a computer-based rainfall-runoff model, such as the WSDOT’s MGSFLOOD model, U.S. EPA’s HSPF model or the Danish Hydraulic Institute’s MIKE SHE model. The continuous runoff hydrographs that result from these models can be used to design infiltration facilities if they are linked with some approach for estimating infiltration rates. In the most general case, these infiltration rates will change with time during the storm event. These transient infiltration rates can be estimated using computer-based groundwater flow models such as USGS's MODFLOW. The continuous runoff hydrographs can also be linked with more simple approaches for estimating infiltration rates, including the steady-state rates described below. It should be noted, however, that these steady-state rates may over- or under-estimate actual time-dependent rates, depending upon site-specific conditions. The continuous-hydrograph approach is generally required to meet water quantity requirements if the infiltration pond is designed with an overflow system that discharges to a stream or surface water body. This method is described in Section 2.2 and Appendix III-B of Volume III of the WDOE manual (2001).

2.2 Choose a trial geometry and estimate depth in the pond

The methodology described below for estimating infiltration rates is based on regression equations to estimate gradients. These equations depend upon the size of the pond, A_{pond}, and upon the depth of water in the pond, D_{pond}. Larger ponds result in smaller gradients and deeper ponds result in higher gradients. This dependency introduces some difficulty or complications into the design process because the gradient is needed to select the size of the pond and the size of the pond is needed to estimate the gradient. An iterative approach is recommended, wherein a trial pond size and depth are assumed, perhaps based on experience at similar sites. The design approach described below can then be used to check the trial design. After the design has been fully developed, the trial pond size should be compared to the design pond calculated below and revised as necessary.

The initial trial geometry, including length, width, and depth of the facility, will generally be based on the characteristics of the property or site that is proposed for the infiltration facility. Computer simulations and observations suggest that elongated facilities with a larger fraction of side area to bottom area may result in higher infiltration rates. Facilities that result in deeper water depths within the facilities also result in higher infiltration rates. If possible, trial geometries should reflect these results. The volume of the infiltration facility for trial geometry should be based on the design volume, V_{design}, described in Section 2.1. The margin or factor safety that is used in this calculation should reflect the site-specific consequences that would occur if the actual stormwater volume exceeds the design volume.

2.3 Perform subsurface site characterization and data collection

The WDOE (2001) stormwater manual includes recommendations for subsurface site characterization and data collection activities that are relevant for the design procedures described in this manual. These recommendations are summarized in Table 2. The WDOE recommendations related to depths of soil borings or test pits should be viewed as minimum requirements. The results of computer simulations described in Massmann et al. (2003) suggest that relatively deep features in the subsurface may affect the performance of infiltration ponds and trenches. The identification of small-scale layering and stratigraphy can be particularly important in terms of the potential for groundwater mounding. Continuous and careful sampling is recommended to depths at least as large as the WDOE requirements.

2.4 Estimate saturated hydraulic conductivity from soil information, laboratory tests, or field measurements

A variety of methods can be used to estimate saturated hydraulic conductivity. These methods, which are summarized below, include estimates based on grain size information, laboratory permeameter tests, air conductivity measurements, infiltrometer tests, and pilot infiltration tests. Some of the advantages and disadvantages of these various methods are summarized in Table 3.

2.4.1 Estimate values based on grain size information

Given the requirements for soil sampling and grain size analyses summarized in Table 2, saturated hydraulic conductivity estimates should be developed based on grain size distributions for all layers encountered during the site characterization phase. These estimates from soil gradation data are more reliable than estimates based solely on USDA soil type.

A variety of methods are available for estimating saturated hydraulic conductivity values from grain size information. One of the most simple and most commonly used approaches is the Hazen equation:

$$K_s = Cd_{10}^2 \qquad (2.1)$$

where K_s is the saturated hydraulic conductivity, C is a conversion coefficient, and d_{10} is the grain size for which 10% of the sample is more fine (10% of the soil particles have grain diameters smaller than d_{10}). For K_{sat} in units of cm/s and for D_{10} in units of mm, the coefficient, C, is approximately 1.

A second approach for estimating saturated hydraulic conductivities for soils typical of western Washington was proposed by Massmann et al. (2003):

$$\log_{10}(K_s) = -1.57 + 1.90d_{10} + 0.015d_{60} - 0.013d_{90} - 2.08\mathrm{f}_{\mathrm{fines}} \qquad (2.2)$$

where K_s is in units of cm/sec, d_{60} and d_{90} are the grain sizes in mm for which 60% and 90% of the sample is more fine and f_{fines} is the fraction of the soil (by weight) that passes the number-200 sieve. This approach is based on a comparison of hydraulic conductivity estimates from air permeability tests with grain size characteristics. Other regression relationships between saturated hydraulic conductivity and grain size distributions are available from the literature (e.g. Freeze and Cherry, 1979; Fetter, 1994; Rawls et al., 1982; Rawls and Brakensiek, 1985).

Grain size samples can also be used to estimate hydraulic conductivity through air conductivity tests (Massmann et al., 2003) using the following relationship:

$$K_s = C_f K_{air} \qquad (2.3)$$

The correction factor C_f, is dependent upon the viscosity and density of air and water. If the air conductivity is measured at laboratory temperatures, and if the hydraulic conductivity is also for laboratory temperatures, then the correction factor is equal to 15. This is based on an assumed laboratory temperature of 20° C. If the air conductivity is measured at laboratory temperatures, and if the hydraulic conductivity is for field temperatures, then the correction factor is approximately 11.5. This is based on an assumed field temperature of 10° C. A simple apparatus for estimating air conductivity from grain size samples is described in Massmann and Johnson (2001) and Massmann et al. (2003).

It should be noted that the estimates given by equations (2.1) through (2.3) should be viewed as "order-of-magnitude" estimates. If measurements of hydraulic conductivity are available from laboratory or field tests (as described in sections 2.3.2 and 2.3.3), these data should be weighed more heavily in selecting values of hydraulic conductivity for design purposes.

2.4.2 Estimate values based on laboratory tests

The grain size methods described in the previous section will give order-of-magnitude estimates for hydraulic conductivity for soils that are relatively coarse-grained (sands and some silty sands). For more fine-grained soils, these methods are prone to significant error. Laboratory saturated hydraulic conductivity tests are recommended for fine-grained soils or whenever feasible. These tests should be conducted on samples that are compacted to a density similar to what is anticipated for actual subsurface conditions. Fixed-head and falling-head tests using both rigid-wall and flexible-wall permeameters are commonly used in the geotechnical laboratories. If this type of data is available, it should be considered in the design process.

2.4.3 Estimate values based on field tests

If data from field tests of saturated hydraulic conductivity are available, these should be considered in the design. These field tests typically include single-ring and double-ring infiltrometer tests or pilot infiltration tests (PIT) similar to what is described in Appendix V-B in Volume V of the WDOE stormwater manual (2001).

Recent developments have also shown that field-tests of air conductivity can also be used to estimate saturated hydraulic conductivity values, in much the same way as laboratory tests. Descriptions of these types of field tests are included in Iverson et al (2001a and 2001b), Fish and Koppi (1994), Liang et al. (1996), and Seyfried and Murdock (1997).

2.4.4 Incorporate hydraulic conductivity estimates for layered soils.

In many cases, estimated saturated hydraulic conductivity values are derived from discrete soil samples collected from several depths at several different horizontal locations beneath each of the infiltration ponds. The goal should be to collect at least one sample from each discrete layer at each horizontal location. Table 4 gives an example of how the data from these multiple samples can be combined to obtain a single "effective" or "equivalent" hydraulic conductivity estimate for an infiltration facility. Soil samples were collected at four different horizontal locations at the example site. At three of these horizontal locations, samples were collected from three different layers. At the fourth location, samples were collected from only two different layers. Hydraulic conductivity values were estimated for each soil sample using the Hazen equation. Hydraulic conductivity estimates from different layers at a single horizontal location can be combined using the harmonic mean:

$$K_{equiv} = \frac{d}{\sum \frac{d_i}{K_i}} \qquad (2.4)$$

where d is the total depth of the soil column, d_i is the thickness of layer "i" in the soil column, and K_i is the saturated hydraulic conductivity of layer "i" in the soil column.

The depth of the soil column, d, would typically include all layers between the pond bottom and the water table. However, for sites with very deep water tables (> 100 feet) where groundwater mounding to the base of the pond is not likely to occur, it is recommended that the total depth of the soil column in Equation 2.4 be limited to approximately 20 times the depth of pond. This is to insure that the most important and relevant layers are included in the hydraulic conductivity calculations. Deep layers that are not likely to affect the infiltration rate near the pond bottom should not be included in Equation 2.4.

The harmonic mean given by equation (2.4) is the appropriate effective hydraulic conductivity for flow that is perpendicular to stratigraphic layers (Freeze and Cherry, 1979). For the example site, these harmonic means range from 6 in/hr for locations 1 and 4 to 12 in/hr for location 3, as shown in the last column in Table 4. The harmonic means for each horizontal location are then averaged to obtain a single estimated hydraulic conductivity for each infiltration pond. This average is equal to 8.7 in/hr for the example site.

The approach described by Equation (2.4) and shown in Table 4 is applicable for combining hydraulic conductivity estimates that are derived from a variety of methods, including grain size analyses, laboratory tests, and field tests. When combining these values, it is important to recognize the layer or which for which the test is relevant.

2.5 Estimate the hydraulic gradient

The infiltration rate from a pond or trench is given by the product of the saturated hydraulic conductivity and the hydraulic gradient. The hydraulic gradient describes the driving forces that cause flow from the infiltration facility. The two primary forces are gravity and capillary suction. The relative importance of these forces and the subsequent gradient depends upon a variety of factors, including duration of the infiltration event, local and regional geology, and depth to groundwater.

In general, there are two cases or end-points for estimating gradients. One end-point is sites with relatively shallow groundwater and the second is sites with relatively deep groundwater. For those sites with thick unsaturated zones, infiltration can be approximated by the Green-Ampt equation (e.g. Chin, 2000). This approach assumes ponded water at the ground surface and a wetting front that extends to some depth, L. The wetting front is assumed to move downward as a sharp interface. The soil is assumed saturated above the wetting front (the water content is assumed equal to the porosity). The water content below the wetting front is assumed equal to some lower initial value. The gradient is approximated by the following expression:

$$gradient \approx \frac{D_{pond} + L + h_{wf}}{L} \qquad (2.5)$$

where D_{pond} is the depth of water in the pond or infiltration facility, L is the depth of the wetting front below the bottom of the pond, and h_{wf} is the average capillary head at the wetting front, with units of length. For infiltration ponds and trenches, the average capillary head, h_{wf}, will be small relative to the depth of water in the facility and the depth of the wetting front so this term can be dropped from equation (2.5).

The term "L" in equation (2.5) represents the depth of the wetting front. Because this changes with time as water infiltrates at the ground surface, the gradient also changes with time. The gradient will start out at some value significantly greater than 1 and will approach 1 as the wetting front moves downward. For most infiltration events, the gradient will reach a value of 1 relatively quickly as compared to the duration of the event (e.g. Massmann et al., 2003). A gradient of 1 would be appropriate for these cases. For very short infiltration events or for soils that are relatively fine-grained (e.g. sandy loam), a gradient of 1.5 may be justified.

For the shallow groundwater sites, the possibility of groundwater mounding must be considered in designing infiltration facilities. This mounding will reduce the hydraulic gradient to a value that is often significantly less than 1.0, and the infiltration rate may be much less than the saturated hydraulic conductivity. This is a very important concept and one that is overlooked in design approaches in which infiltration rates are estimated solely on the basis of soil types or saturated hydraulic conductivity estimates. It should be noted that mounding may also result if perched water-table conditions occur due to low-permeability layers beneath site and above the water table. (In very general terms, a layer would be characterized as "low-permeability" in this context if the estimated hydraulic conductivity of the layer is less than 10% of the hydraulic conductivity assigned to the overlying materials and if the hydraulic conductivity of this layer is less than the infiltration rate from the pond. As a first approximation, a layer could be characterized as low permeability in this context if it is less than 0.5 inches per hour and if it is less than 10% of the overlying materials.)

For deep groundwater sites, where the effects of mounding will generally be small, the gradient will not typically be reduced by infiltration from the facility. For these deep sites, the gradient will be approximately equal to 1.0, as described by the Green-Ampt equation. The approach described below uses regression equations to estimate the hydraulic gradient. These equations were developed based on computer simulations described in Massmann et al. (2003) for sites where water table mounds will develop. These regression equations can also be applied to sites with deep groundwater by limiting the gradient to a maximum value of 1.0.

Massmann et al. (2003) report the results of computer simulations that were based on the geometry and observed geology beneath the Lacey-Lid infiltration pond in Thurston County, Wash. as described in the *Water Resources Investigations Report 92-4109* (Drost et al., 1999). Based on the results of these computer simulations, the effective gradient under steady-state conditions beneath a medium-sized infiltration facility can be approximated with the following expression:

$$\text{gradient} = \text{i} \approx \frac{D_{wt} + D_{pond}}{138.62(K^{0.1})} CF_{size} \tag{2.6}$$

where K is saturated hydraulic conductivity in feet/day, D_{wt} is the depth in feet from the base of the infiltration facility to the water table or to the first low-permeability layer, and D_{pond} is the depth of water in the pond, in feet. The regression equation given above was developed using computer simulations for ponds with infiltration rates that ranged from 0.2 to 20 inches per hour (saturated K values from 1.5 to 150 in/hour). The data that were used to develop the regression equation given by Equation 2.6 and the approach used to select the form of the equation are described in Massmann et al. (2003) and summarized in Table 5.

The correction for pond size, CF_{size}, is given by the following expression:

$$CF_{size} = 0.73(A_{pond})^{-0.76} \tag{2.7}$$

where

CF_{size} = correction factor for size of the pond

A_{pond} = area of the pond bottom in acres

This expression for correction factor was developed for ponds with bottom areas between 0.6 and 6 acres in size. Figure 3 shows the relationship between Equation (2.7) and the calculated correction factor based on computer simulations. For small ponds (ponds with area equal to 2/3 acre), the correction factor is equal to 1.0. For large ponds (ponds with area equal to 6 acres), the correction factor is 0.2.

Massmann et al. (2003) also report the results of computer simulations for infiltration trenches in which the length dimension is much larger than the width. Based on the results of these computer simulations, the effective gradient under steady-state conditions beneath an infiltration trench can be approximated with the following expression:

$$\text{gradient} = \text{i} \approx \frac{D_{wt} + D_{trench}}{78(K^{0.05})} \tag{2.8}$$

where K is saturated hydraulic conductivity in feet/day, D_{wt} is the depth in feet from the base of the infiltration trench to the water table or to the first low-permeability layer, and D_{trench} is the depth of water in the trench, in feet. The regression equation given by Equation 2.8 was developed using computer simulations for trenches with infiltration rates that ranged from 0.2 to 20 inches per hour (saturated K values from 1.5 to 150 in/hour), as described in Massmann et al. (2003). There is no correction factor required for trench size.

The relationships given in Equation 2.6 and 2.8 are based on estimates of infiltration rates derived from the computer simulations presented in Massmann et al. (2003). These relationships were derived by estimating infiltration rates as a function of depth to groundwater using the computer model. The results of these computer simulations allowed a relationship to be developed between the infiltration rate and the depth to groundwater. As the depth to groundwater was decreased in the computer model, the infiltration rate also decreased. This observed relationship can be incorporated into Darcy's law through the gradient term. As the depth to groundwater decreases, the gradient also decreases. Equations 2.6 and 2.8 provide estimates for relating the depth to groundwater and the gradient.

Figure 4 compares the gradients for ponds estimated using Equation (2.6) with the gradients calculated using the model. The data shown with circles is the data used to develop Equation (2.6), as described in Massmann et al. (2003). Additional computer simulations were then conducted using larger ponds and ponds with deeper water tables to evaluate the robustness of the regression equation. The open squares show the results of this verification. The data and computer models used in the regressions are more fully described in Massmann et al. (2003). Figure 4 shows that the regression equation provides very good estimates of the modeled systems with gradients less than approximately 0.2. The equation also provides reasonable estimates for gradients for systems with deeper water tables. Furthermore, the regression equation is conservative for the deep water table ponds in that the estimated gradients are smaller than the model predictions. Figure 5 compares gradients for trenches calculated using Equation 2.8 with the gradients calculated using the model. These results also show that the regression equation provides good estimates of the modeled systems with gradients less than approximately 0.3.

It is important to note that the relationships described in Equations (2.6) through (2.8) are approximations that were derived from a set of computer simulations for a particular facility in a particular hydrogeologic system. It is believed that these equations are representative of facilities at sites where the depth to groundwater is from several feet to approximately 100 feet. For systems deeper than 100 feet, where the Green-Ampt equation will provide a better representation of the infiltration processes, a gradient of 1.0 is recommended in lieu of the gradients given by Equations (2.6) and (2.8).

It should also be noted that Equations (2.6) and (2.8) do not incorporate the effects of perched water tables that result beneath the infiltration facility. Under some hydrogeologic conditions, water-table mounding may occur above low-permeability layers that lie above the "normal" or regional water table. This mounding can result in an unsaturated zone between the saturated, perched water and the normal or regional water table. Under these conditions, the appropriate depth that should be used in Equations (2.6) and (2.8) is not the depth to the regional water table, but rather the depth to the low-permeability layer that may cause perched conditions. In very general terms, a layer could be characterized as "low-permeability" in this context if the estimated hydraulic conductivity of the layer is less than 10% of the hydraulic conductivity assigned to the overlying materials and if the hydraulic conductivity of this layer is less than the infiltration rate from the pond. As a first approximation, a layer could be characterized as low permeability in this context if it is less than 0.5 inches per hour and if it is less than 10% of the overlying materials.

Equation (2.6) shows that the gradient depends upon the size of the pond, A_{pond}, upon the depth of water in the pond, D_{pond}. Larger ponds result in smaller gradients and deeper ponds result in higher gradients. This introduces some difficulty in that the gradient is needed to select the size of the pond and the size of the pond is needed to estimate the gradient. An iterative approach is recommended, wherein a trial pond size and depth is assumed, perhaps based on experience in similar sites. The trial pond size is used in Equation (2.7) to get a pond correction factor and the depth of water is used in Equation (2.6) to estimate the gradient. The trial pond size should be compared to the actual pond size calculated below to determine if a revised correction factor is required.

2.6 Estimate the infiltration rate by multiplying gradient and hydraulic conductivity.

Based on Darcy's law, the infiltration rate can be estimated by multiplying the saturated hydraulic conductivity from Section 2.4 with the hydraulic gradient determined in section 2.5:

$$f = K\left(\frac{dh}{dz}\right) = Ki \qquad (2.9)$$

where f is the specific discharge or infiltration rate of water through a unit cross-section of the infiltration facility (L/t), K is the hydraulic conductivity (L/t), dh/dz is the hydraulic gradient (L/L), and "*i*" is a "short-hand" notation for the gradient (given by Equation (2.6) or (2.8) for ponds and trenches at sites with shallow water tables, or a value of approximately 1 for ponds and trenches at sites with deep water tables).

2.7 Apply correction factors for biofouling, siltation, and pond geometry

The infiltration rate given in Equation (2.9) was developed assuming that the hydraulic conductivity of the soil beneath the infiltration facility will remain equal to the value measured in the field or laboratory tests or estimated using soil information. Depending upon the level of pre-treatment and the maintenance program that is put in place at the facility, the long-term infiltration rates may be reduced significantly by factors such as siltation and biofouling. Siltation is more likely to occur if there is not sufficient pre-treatment of the storm water or in locations where the drainage basin is prone to erosion because of recent land disturbances or steep slopes. Biofouling is more likely to occur if the pond is located beneath trees and other vegetation or in shaded locations.

If effective pre-treatment and reliable long-term maintenance cannot be guaranteed, the infiltration rates used in Equation 2.9 should be reduced. Table 6 gives infiltration rate reduction factors to account for biofouling and siltation effects for infiltration ponds and trenches. These factors, which are somewhat subjective, were developed based on the field observations and computer simulations reported by Massmann et al (2003). The infiltration rates calculated using Equation (2.9) should be multiplied by these correction factors to account for the effects of siltation and biofouling.

Although siltation and biofouling may be less prevalent in infiltration trenches as compared to infiltration ponds, field data have not been collected that would allow correction factors to be estimated for these trenches. The conservative approach would be to use the same correction factors for trenches as for ponds. However, the computer simulation results described in Massmann et al. (2003) suggest that reductions in hydraulic conductivity due to bottom clogging from siltation and biofouling may have relatively small effects on overall infiltration rates and gradients for trenches. This is because of the larger amounts of lateral flow that occurs in trenches relative to ponds. Reductions in vertical flow from the bottom of the trench are offset by increases in lateral flow, particularly for trenches with deeper water levels. Based on these results, it may be more appropriate to use correction factors that are included Column D in Table 6.

Computer simulations described in Massmann et al. (2003) also suggest that ponds with large aspect ratios (defined as pond length divided by pond width) have higher infiltration rates than ponds with lower aspect ratios. The data in Table 7 can be used to develop an equation for correction factor that can be used to account for these results:

$$CF_{aspect} = 0.02A_{ratio} + 0.98 \qquad (2.10)$$

where A_{ratio} is the aspect ratio for the pond (length/width). In no case should the correction factor for aspect ratio be greater than 1.4.

The correction factors for siltation and biofouling and for aspect ratio are multiplied by the infiltration rate given by Equation (2.9):

$$f_{corr} = (CF_{silt/bio})(CF_{aspect})f = (CF_{silt/bio})(CF_{aspect})Ki \qquad (2.11)$$

where $CF_{silt/bio}$, is the correction factor for siltation and biofouling, CF_{aspect} is the correction factor for aspect ratio, and f is the "uncorrected" infiltration rate given by Equation 2.9. (Note: The aspect ratio correction is not applied to trench configurations.)

Once an infiltration rate has been estimated using Equation (2.11), it should be compared with rates from the literature, from design manuals, and from observations at other similar facilities in similar hydrogeologic environments for verification purposes. For example, the rates can be compared to those rates given in the WDOE stormwater manual (see Tables 8 through 10 below). Table 8 gives rates that are based on USDA soil type, Table 9 gives rates that are based on soil gradation analyses, and Table 10 gives correction factors that are used to reduce infiltration rate estimates from field-scale tests, including PIT tests.

In many instances, the infiltration rates estimated using Equation (2.11) will be significantly larger than the rates given in Tables 8 through 10. The rates given in Tables 8 through 10 from the WDOE manual were developed based on observations at field sites in western Washington. Many of these sites have reduced infiltration rates due to maintenance and design issues, as described in Massmann et al. (2003) and Wiltsie (1998).   The rates in Tables 8 through 10 were also developed for sites with relatively shallow water tables and will likely be overly conservative for sites with deep water tables and thick unsaturated zones. Figure 6 illustrates the range of values for infiltration rates that can be expected based on maintenance and design practices. This figure includes both the rates recommended in the WDOE manual and actual rates measured in full scale facilities in Western Washington with shallow ground water. The shaded range in this figure can be particularly useful for making the comparison described above.

The infiltration rates given by Equation (2.11) may, in fact, be larger or smaller than the rates given in Tables 8 through 10 or in Figure 6 for reasons that are reality-based.   The important question is whether an explanation can be developed for the differences. For example, if the value given by Equation (2.11) is for a site with coarse gravel and a deep water table, it may be significantly larger than the rates in the figures, tables, or other databases. Similarly, if the site is located in low-permeability materials with a shallow water table and no pretreatment, the value from Equation (2.11) may be smaller than previously-observed values.

Other safety factors could also be applied to the infiltration rate at this step in exceptional circumstances to account for other uncertainties. It should be noted that there are "built-in" safety factors in the methodology described above related to the hydrologic analyses used to estimate the volume of infiltrated water, V_{design}. These safety factors are related to the design storm that is used to generate runoff for the infiltration facility. Additional safety factors are built into Table 6, as the factors provided in that table were derived from the shallow ground water sites in Western Washington mentioned previously. In effect, these factors were derived to yield similar results to what is provided in Table 9 for the shallow ground water conditions from which the values in Table 9 were derived. The WDOE manual specifies that the values in Table 9 may be used without the application of additional reduction or safety factors. The method proposed herein (i.e., Equation 2.11) is consistent with this philosophy.

The infiltration rate given by Equation (2.11) can be combined with the gradient equation given by Equation (2.6) to obtain a relationship between hydraulic conductivity, gradient, and infiltration for infiltration ponds:

$$\mathrm{Q_{ponds}} = K\mathrm{iA_{total}} \approx K\left[\frac{D_{wt} + D_{pond}}{138.62(K^{0.1})} CF_{size}\right](CF_{silt/bio})(CF_{aspect})A_{total} \qquad (2.12)$$

where, A_{total} is as defined in Section 2.8.

A similar equation can be developed for trenches by combining Equations (2.8) and (2.11):

$$\mathrm{Q_{trench}} = K\mathrm{iA_{total}} \approx K\left[\frac{D_{wt} + D_{trench}}{78(K^{0.05})}\right](CF_{silt/bio})A_{trench} \qquad (2.13)$$

where, A_{trench} is the wetted area of the trench sides and bottom.

2.8 Design approaches for single-event hydrographs

In most cases, flow from the infiltration facilities will occur through both the sides and the bottom of the facility. It may be useful in some instances to quantify the magnitude of these two components in order to assess the effects of bottom plugging and other maintenance issues. Based on observations and computer simulations described in Massmann et al. (2003), horizontal flow from facilities may be significant and is sensitive to the average depth of water during the infiltration event.

The total flow out the sides and bottom of the facility can be estimated to a first approximation with the following expressions:

$$Q = Q_{sides} + Q_{bottom} = f_{corr}(A_{sides} + A_{bottom}) = f_{corr}(A_{total}) \tag{2.14}$$

where f is the infiltration rate of water through a unit cross-section of the infiltration facility (L/t) estimated using Equation (2.11), Q is the volumetric flow rate (L^3/t), A_{sides} and is the cross-sectional area of the submerged pond sides in a vertical plane, A_{bottom} is the cross-sectional area of the pond bottom in a horizontal plane, and A_{total} is the total area of both sides and bottom. In the general case, the value for both A_{sides} and A_{bottom} will depend upon the depth of water in the facility. Reasonable first approximations can be derived using values based on one-half the maximum depth of water for the design storm. As a minimum, the total flow rate, Q, times the required draining time, T_{req} (e.g. from Table 1) should be greater than the design volume, V_{design}.:

$$(Q)T_{req} \geq V_{design} \tag{2.15}$$

2.9 Design approaches for continuous hydrographs

The required infiltration rates given by equation (2.15) were developed based on the assumption that the complete design flow volume arrives “instantaneously” at the infiltration facility. The WDOE storm-water manual (2001) specifies that the infiltration facility must be designed to drain completely within 24 hours **after the flow to it has stopped**. For the 24-hour design event, flow will not stop arriving to the infiltration facility until at least 24 hours after the storm begins. The WDOE manual effectively allows at least 48 hours from the beginning of the storm event before the water must be infiltrated.

For typical inflow hydrographs, similar to what is shown in Figure 2, most of the stormwater arrives relatively early in the storm event, which allows more than 24 hours for it to become infiltrated. The methodologies described by Equation (2.15) do not include this "extra" time. More importantly, the single-event hydrograph does not directly account for the effects of a long sequence of multiple precipitation events.

Continuous flow models such as WSDOT's MGSFLOOD model or U.S. EPA's HSPF can be used to incorporate the transient effects described by continuous hydrographs. One approach for accomplishing this is to include the infiltration facility in the runoff or flow models using a stage-discharge relationship. This stage-discharge relationship describes the flow rate as a function of the depth of water in the pond or trench. The infiltration equations given by Equations (2.12) and (2.13) can be used for these purposes. These equations effectively provide stage-discharge relationships for ponds and trenches. Both the gradient term and the area term in Equations (2.12) and (2.13) depend upon the depth or stage of water in the pond or trench. These stage discharge relationships can be directly implemented within the continuous flow models to evaluate the change in depth in the facility with time.

The typical design approach for using continuous hydrographs for facilities without overflow features would be to select a pond geometry that would provide a minimum freeboard for the continuous design hydrograph. The magnitude of this minimum freeboard would be dependent upon the consequences of overflow from the facility.

For facilities that include overflow features or design components, the design approach would be to select the pond geometry to meet the performance objective for water quantity considerations given by Minimum Technical Requirement #7 (MTR #7) in the WDOE Stormwater Management Model (Volume I, Chapter 2, page 2-31). This requirement specifies that stormwater discharges to streams must mimic certain aspects of pre-developed conditions, as described in Section 1.

2.10 Consider computer simulations to refine design

More sophisticated computer-based simulations should be considered for many sites. It is likely that these simulations will result in facilities that are less over-designed than the facilities that would result from infiltration rates given in the WDOE stormwater manual (2001). These simulations should be especially considered for the following situations:

1) Sites with significant heterogeneity and stratigraphy, particularly sites where subsurface lateral flow through higher-permeability strata may be extensive.
2) Sites where transient effects of inflow rates are likely to be important, as discussed in Section 2.9.
3) Sites with water table depths or depths to low-permeability layers that are greater than 100 feet. These types of sites were not included in the computer simulations described in Massmann et al. (2003) and the regression equation given by Equations (2.6) and (2.8) may significantly under-estimate the actual gradient.

Steady-state, saturated simulations similar to what can be developed using the USGS MODFLOW computer code will generally give conservative results that are more realistic than the procedures described in this manual. For particularly important sites, transient, unsaturated models may be warranted, similar to what is described in Massmann et al. (2003).

2.11 Post design-evaluations

Full-scale tests should be conducted at all sites on a periodic basis where possible. If a source of water is available (e.g. nearby fire hydrants or water trucks), these tests should be conducted using controlled and measured inflow rates that result in significant ponding in the facility. If water sources are not available, inflow rates should be monitored if at all possible.

By monitoring inflow rates, relationships can be developed that give infiltration rates as a function of stage or water level in the facility. These types of relationships are particularly valuable if computer-simulations are used to evaluate the design performance, or if continuous hydrographs are used in the design approach. Appendix A describes methods for conducting and analyzing data from the full-scale tests with known inflow rates.

In cases where the full-scale tests indicate infiltration rates that are significantly less than the design rates, the facility may need to be modified. If the lower rates are expected to be caused by soil plugging or bio-fouling, then remediation of the existing pond may be possible. For some sites, particularly those where the lower rates are due to unexpectedly high groundwater levels, there may be little that can be done, other than increasing the areal extent of the facility or designing an overflow system. The approaches described in this manual can be used to re-evaluate these facility retrofit options. In many instances, more refined and sophisticated analyses will be warranted.

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Table 1 – Example water quality performance objectives for infiltration facilities

Agency	Recommended or allowable storage time, T_{req}	Reference
Federal Highway Administration	**72 hours**	*Urban Design Drainage Manual*, Hydrologic Engineering Circular No. 22, Washington D.C., 1996.
Maryland Department of the Environment	**48 hours**	*Maryland Stormwater Design Manual*, Center for Watershed Protection , Annapolis, MD 1998.
California Department of Transportation	**24 hours**	*Stormwater quality handbook*, Project Training and Design Guide, Sacramento, CA, 2000.
Washington Department of Ecology	**24 hours**	*Stormwater Management Manual for Western Washington*, Publication 99-13, Olympia, WA, 2001.

Table 2 – WDOE requirements for subsurface characterization at infiltration facilities (from WDOE, 2001)

WDOE (2001) Requirements	Comments and Recommendations
Subsurface explorations (test holes or test pits) to a depth below the base of the infiltration facility of at least 5 times the maximum design depth of ponded water proposed for the infiltration facility. (p. 3-62)	This should be viewed as a minimum requirement. Infiltration rates from stormwater facilities can be affected by relatively deep features in the groundwater flow system, particularly for sites with subsurface layering. Deeper features may be especially important in western Washington where there are long periods of precipitation that may cause groundwater mounding and where facilities may receive runoff from multiple storms. In many cases, it may be prudent to continue subsurface explorations to depths of 50 feet or greater.
Continuous sampling (representative samples from each soil type and/or unit within the infiltration receptor) to a depth below the base of the infiltration facility of 2.5 times the maximum design ponded water depth, but not less than 6 feet. (p. 3-62)	This should be viewed as a minimum requirement. It may not be conservative, especially for ponds that receive runoff from multiple storms and for sites with subsurface layering. Relatively small-scale layering can affect infiltration performance, even if these features occur relatively deep beneath the facility. Continuous samples and more detailed sampling to greater depth is probably warranted in many cases.
For basins, at least one test pit or test hole per 5,000 ft ^{2} of basin infiltrating surface (in no case less than two per basin) (p. 3-62)	This is a reasonable amount of investigation.
For trenches, at least one test pit or test hole per 50 feet of trench length (in no case less than two per trench). (p. 3-62)	This is a high density of test pits, especially as compared to the sampling frequency for ponds
Prepare detailed logs for each test pit or test hole and a map showing the location of the test pits or test holes. Logs must include at a minimum, depth of pit or hole, soil descriptions, depth to water, presence of stratification. (p. 3-63)	This is a reasonable and appropriate recommendation.

WDOE (2001) Requirements	Comments and Recommendations
As a minimum, one soil grain-size analysis per soil stratum in each test hole shall be performed within 2.5 times the maximum design water depth, but not less than 6 feet.	Grain size analyses should be performed on samples from all strata encountered in the soil borings. Deeper borings are recommended.
Soil characterization for each soil unit (soils of the same texture, color, density, compaction, consolidation and permeability) encountered should include: grain-size distribution, textural class (USDA), percent clay content (include type of clay, if known, color/mottling, and variations and nature of stratification	These are prudent and reasonable recommendations.
Installation of ground water monitoring wells (at least three per infiltration facility, unless the highest ground water level is known to be at least 50 feet below the proposed infiltration facility)	This is a prudent and reasonable recommendation. Note that if wells are installed to 50 feet, continuous or near-continuous soil samples are recommended for the full depth.
Monitor the seasonal ground water levels at the site during at least one wet season.	This is a prudent and reasonable recommendation
Estimate of the volumetric water holding capacity of the infiltration receptor soil.	This is a prudent and reasonable recommendation
Existing ground water flow direction and gradient ,lateral extent of infiltration receptor, horizontal hydraulic conductivity of the saturated zone	These are prudent and reasonable recommendations.
Impact of the infiltration rate and volume at the project site on ground water mounding, flow direction, and water table; and the discharge point or area of the infiltrating water. A ground water mounding analysis should be conducted at all sites where the depth to seasonal ground water table or low permeability stratum is less than 15 feet and the runoff to the infiltration facility is from more than one acre.	Groundwater mounding analyses are prudent. However, the one-acre requirement for a mounding analysis is arbitrary. This should be based on the anticipated depth of stormwater discharge. Low-permeability strata below 15 feet may also affect mounding and should be considered in mounding analyses.

Table 3 – Comparison of methods for estimating saturated hydraulic conductivity values for unconsolidated soils above the water table.

Method	Advantages	Disadvantages
Regressions using grain size information	Inexpensive; rapid sample collection; can often collect many samples to evaluate heterogeneity and layering	Errors may be significant (a factor or 10 or more), especially for soils with large percentages of fine materials or swelling clays
Laboratory permeameter tests	Measures effects of swelling, well-accepted methods and protocols	Typically disturbed samples; requires specialized equipment and expertise
Field infiltrometer tests	Measures unsaturated flow processes; gives in-situ results that reduce the effects of sample disturbance	Time consuming, only possible with near-surface soils that can be exposed in excavations; should be used to estimate saturated hydraulic conductivity and not infiltration rate
Pilot infiltration tests	Gives large-scale estimate that incorporates effects of heterogeneity	Large volumes of water are typically required; only possible with near-surface soils that can be exposed in excavations
Packer permeability tests in boreholes	Allows in-situ measurements at depth, allows specific strata to be tested	Expensive and time-consuming, requires specialized equipment and expertise, large volumes of water may be required
Estimates from air conductivity	No water is required; tests can be done in the field or in the laboratory; relatively rapid test;	Requires specialized equipment and expertise; errors may be significant if swelling occurs; methods and protocols not yet well-accepted

Table 4 – Example calculations for equivalent hydraulic conductivity using the Hazen approximation

Layer	Layer thickness (inches)	d_{10} (mm)	K_w (cm/s)	K_w (in/hr)	$K_{equivalent}$ (in/hr)
1	13	0.05	2.2E-03	3	
1	8	0.08	5.6E-03	8	
1	10	0.18	2.8E-02	40	6
2	9	0.09	7.0E-03	10	
2	6	0.13	1.5E-02	21	
2	17	0.09	7.0E-03	10	11
3	14	0.07	4.3E-03	6	
3	8	0.54	2.5E-01	360	
3	7	0.23	4.6E-02	65	12
4	18	0.06	3.1E-03	4	
4	8	0.34	1.0E-01	143	6
Averages:				**61**	**8.7**

Table 5 - Modeled infiltration rates for different water table elevations (from Massmann et al., 2003)

Depth of Water Table (ft)	**Hydraulic conductivity beneath facility (in/hr)**	**Infiltration Rate**		**Percent of Total Infiltration**	
		cm/hr	**in/hr**	**Bottom**	**Sides**
20	1.5	0.7	0.3	69	31
	2.5	1.0	0.4		
	5	1.8	0.7		
	25	9.2	3.6		
	50	17.7	7.0		
	150	49.4	19.4		
15	1.5	0.5	0.2	68	32
	2.5	0.8	0.3		
	5	1.5	0.6		
	25	7.6	3.0		
	50	14.7	5.8		
	150	41.2	16.2		
10	1.5	0.3	0.1	67	33
	2.5	0.6	0.2		
	5	1.1	0.4		
	25	5.9	2.3		
	50	11.5	4.5		
	150	32.4	12.8		
5	1.5	0.2	0.08	66	34
	2.5	0.3	0.1		
	5	0.7	0.3		
	25	4.1	1.6		
	50	8.1	3.2		
	150	23.1	9.1		
0	1.5	0.02	0.01	66	34
	2.5	0.1	0.04		
	25	2.3	0.9		
	50	4.54	1.8		
	150	13.1	5.2		

Table 6 —Infiltration rate reduction factors to account for effects of biofouling and siltation (from Massmann et al., 2003)

A	B	C	D
Potential for biofouling	**Degree of long-term maintenance and performance monitoring**	**Infiltration rate reduction factor**	
		Ponds	**Trenches**
Low	Average to high	0.9	0.9
Low	Low	0.6	0.8
High	Average to high	0.5	0.75
High	Low	0.2	0.6

Table 7 —Infiltration rates for different pond perimeters (from Massmann et al., 2003)

Aspect ratio	Pond Dimensions (ft x ft)	Correction factor for aspect ratio	Flow out sides (%)
1	170 x 170	1.0	30.7
2	120 x 240	1.02	31.4
8	480 x 60	1.09	35.5

Table 8 — Recommended infiltration rates based on USDA soil textural classification (from Table 3.7 in Vol. III and Table 7.1 in Vol. V, WDOE, 2001)

	Short-Term Infiltration Rate, in/hr (cm/hr)	Correction Factor	Estimated Long-Term Infiltration Rate, in/hr (cm/hr)
Clean sandy gravels and gravelly sands (i.e., 90% of the total soil sample is retained in the #10 sieve)	20 (50)	2 (5)	10 (25)
Sand	8 (20)	4 (10)	2 (5)
Loamy Sand	2 (5)	4 (10)	0.5 (1.3)
Sandy Loam	1 (2.5)	4 (10)	0.25 (0.64)
Loam	0.5 (1.3)	4 (10)	0.13 (0.33)

Table 9—Recommended infiltration rates based on ASTM gradation testing (from Table 3.8 in Vol. III and Table 7.2 in Vol. V, WDOE, 2001)

D10 Size from ASTM D422 Soil Gradation Test mm	**Estimated Long-Term Infiltration Rate, in/hr (cm/hr)**
>0.4	9 (23)
0.3	6.5 (16.5)
0.2	3.5 (8.9)
0.1	2.0 (5)
0.05	0.8 (2)

Table 10—Correction factors to be used with in-situ infiltration measurements to estimate long-term design infiltration rates (from Table 3.9 in Vol. III and Table 7.3 in Vol. V, WDOE, 2001)

Issue	**Partial Correction Factor**
Site variability and number of locations tested	CFv = 1.5 to 6
Degree of long-term maintenance to prevent Siltation and bio-buildup	CFm = 2 to 6
Degree of influent control to prevent siltation and bio-buildup	CFi = 2 to 6
Total correction factor	CF = CFv + CFm + CFi

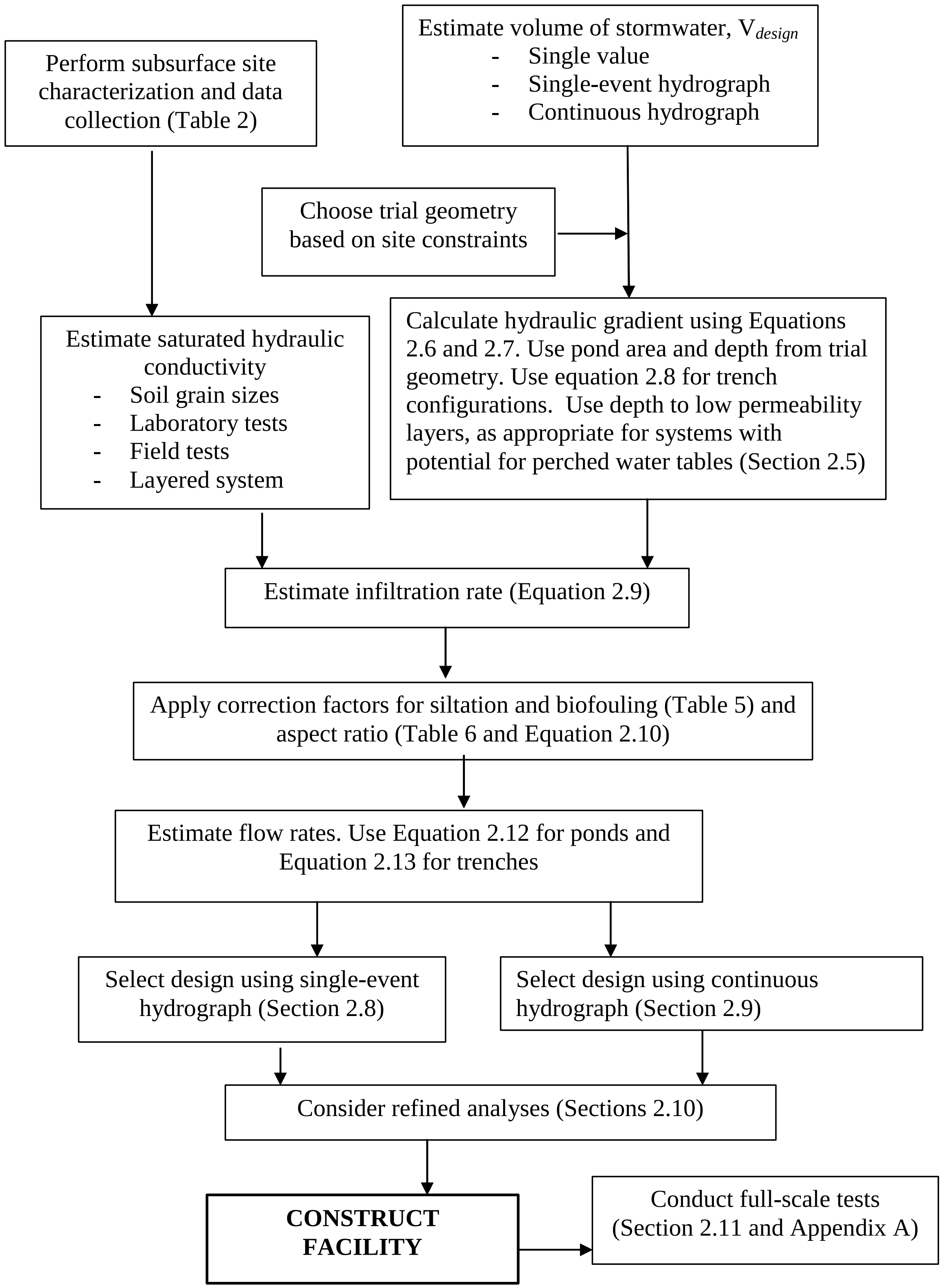

Figure 1 – Flow chart summarizing design approach for ponds

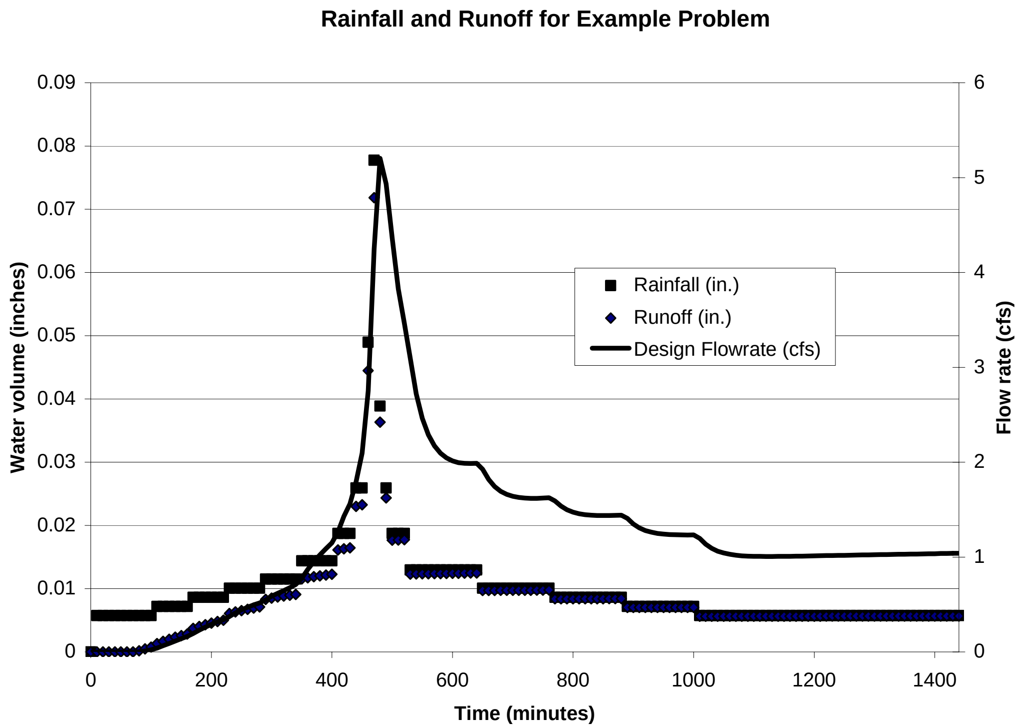

Figure 2– Example single-event hydrograph used for infiltration pond design

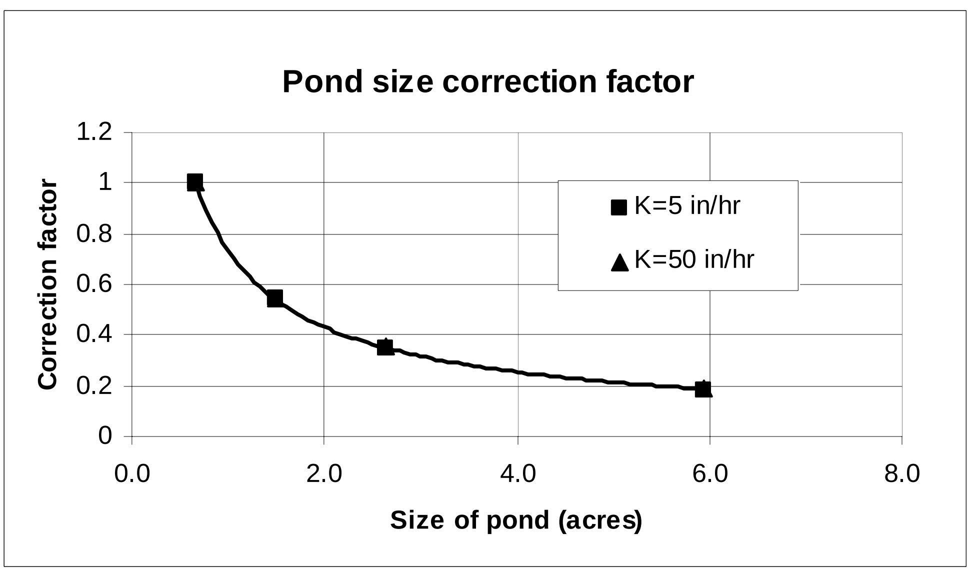

Figure 3– Correction factors for pond size given by Equation (2.6) (Massmann et al., 2003)

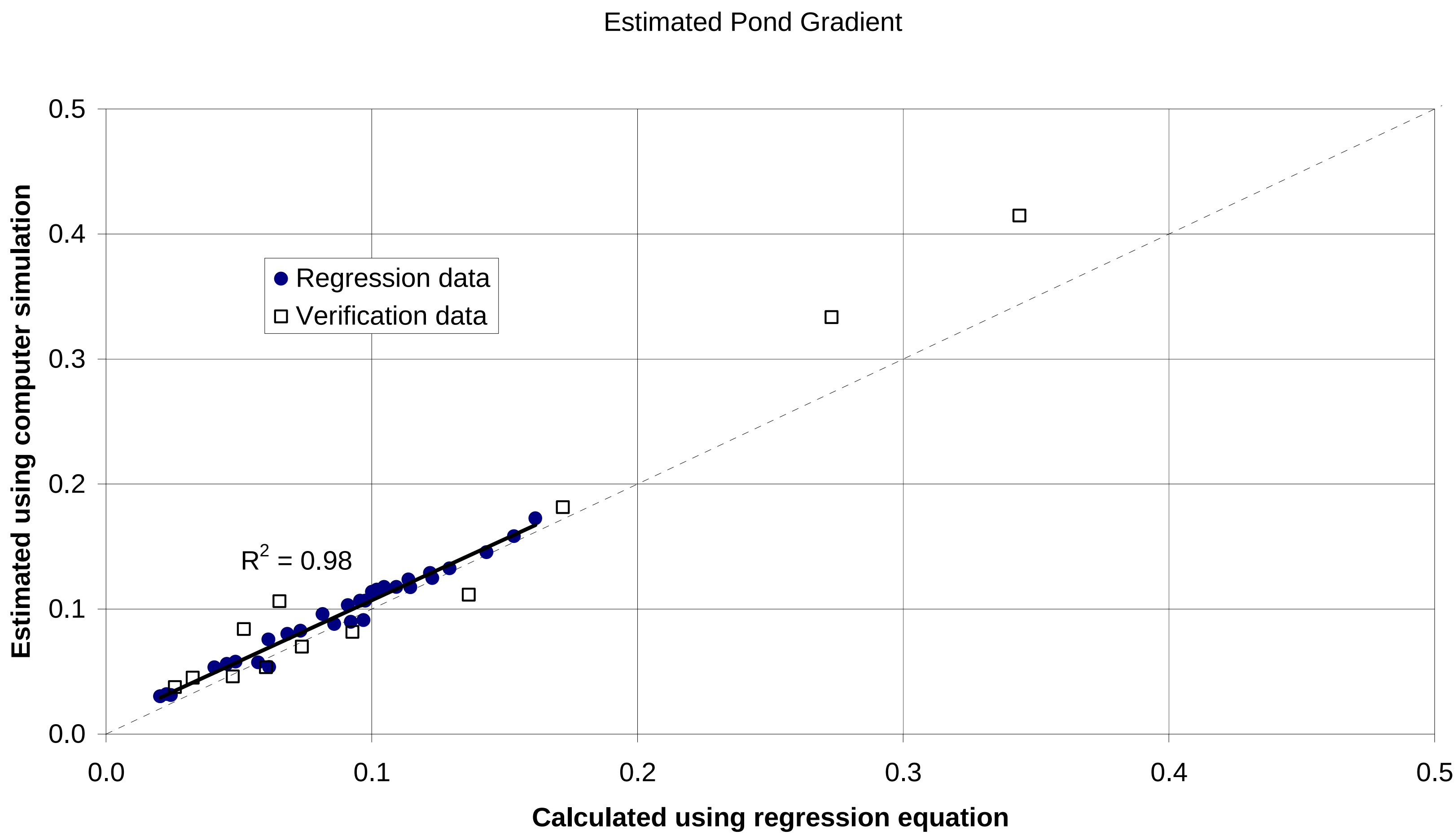

Figure 4 – Comparison of gradients for ponds calculated using Equation (2.5) with gradients simulated using computer models (Massmann et al., 2003)

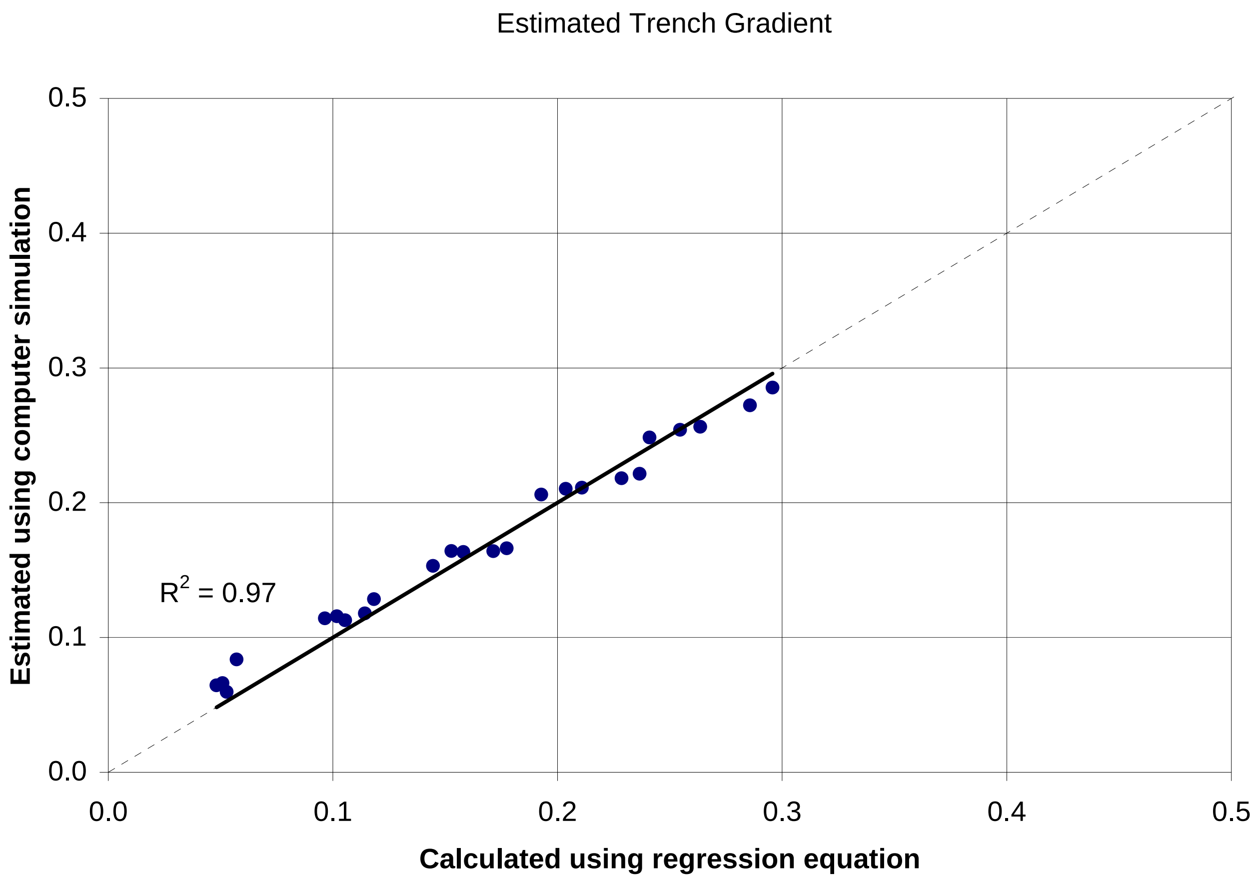

Figure 5– Comparison of gradients for trenches calculated using Equation (2.7) with gradients simulated using computer models (Massmann et al., 2003)

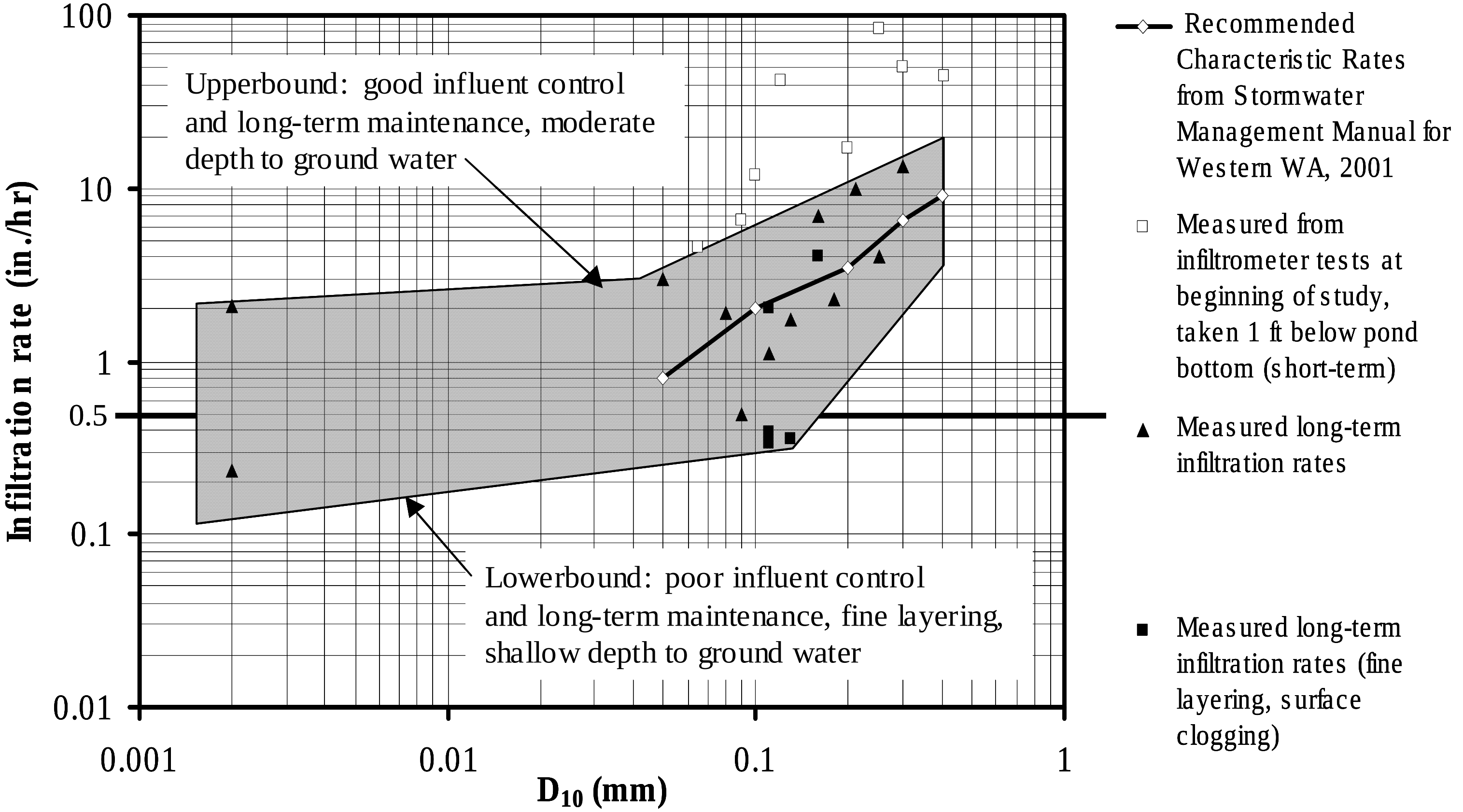

Figure 6 – Comparison of small scale, short-term infiltrometer tests with long-term, full-scale observations (Massmann et al., 2003).

APPENDIX A: METHODS FOR CONDUCTING AND ANALYZING FULL-SCALE INFILTRATION TESTS

This appendix describes methods that can be used to complete full-scale infiltration tests to estimate infiltration rates for ponds and similar infiltration facilities. The methods used to conduct these tests are described in Section A.1 and the approach used to analyze the data is described in Section A.2. An example is given in Section A.3. For more details on implementing these tests, see Chapter 4, Massmann et al. (2003).

Compared to other field and laboratory methods, full-scale tests provide the most reliable estimates of how infiltration ponds will perform. These tests, which are often termed "flood tests," are conducted by discharging water into a pond and measuring the change in water level in the pond as a function of time. Data are collected both during the filling stage of the test and during the draining stage after the discharge into the pond has been stopped. The primary advantage of these full-scale tests is that the infiltration rate is measured directly. The approach does not require modeling or significant analytical tools to estimate or infer infiltration rate from some other measured parameter. Sampling errors that can be introduced with small-scale measurements in heterogeneous soils are avoided. The primary disadvantage is that these tests require a source of water that can be delivered at a high enough flow rate to cause ponding within the facility.

A.1 Field Methods for the Full-Scale Tests

A.1.1 Measure pond geometry

Surveying measurements should be taken to calculate the pond geometry. If practical, elevation measurements should be collected along transects with a horizontal spacing between measurements of approximately four to six feet, depending upon the site. The length and width of the pond bottom can be measured with a steel tape measurer. The side slopes (change in elevation divided by change in horizontal distance) should be recorded.

A.1.2. Complete pre-test calculations

The objective is to estimate the volume of water that would be required to achieve ponding in the infiltration facility. This is accomplished by defining both the discharge and the duration of test. The following steps are used:

1. Develop preliminary estimates of infiltration rates based on soil type or other available data.
2. Calculate the minimum discharge rate that would be required to achieve ponding: Q_{min} = Pond area* estimated infiltration rate
3. Choose a desired maximum depth in the pond (> 2 feet if possible)
4. Choose a desired duration for the inflow portion of the test (< 8 hrs)
5. Estimate the discharge rate to achieve a prescribed depth of water in a prescribed time: Q_{test} = *(area* desired depth /desired inflow time)* + Q_{min}
6. Multiply Q_{test} by some factor of safety (~1.5 to 3)
7. Estimate test time as *(area* desired depth)*/Q_{min}

A.1.3 Install pressure transducers

At least one pressure transducer is recommended, although staff measurements can also be used if necessary. Transducers should be secured so they are not disturbed during the test, as shown in Figure A-1. Set sampling interval based on test duration and discharge duration.

A.1.4 Perform flood test and import data to spreadsheet

A.2 Data Analysis for the Full-Scale Tests

The rate of infiltration is estimated using data from full-scale tests by performing a water balance on the infiltration pond. The following expression gives the volume of water that infiltrates during a time interval dt:

$$V_{\inf} = Q_{in}(dt) - A_{surf}(dz) \qquad \text{(A.1)}$$

where V_{inf} is the volume of water that infiltrates during the time interval (L^3), Q_{in} is the flow rate into the pond (L^3/t), A_{surf} is the area of the pond surface (L^2), and dz is the change in water depth during the time interval (L). The sign convention that is used in equation A.1 is that a positive dz denotes an increase in the depth of the water in the pond.

Except for ponds with rectangular cross-sections, the area of the water surface, A_{surf}, will be a function of the depth of the water in the pond, dz. The surface area for rectangular ponds can be calculated using the following expression:

$$A_{surf} = \left(L * W\right) + \frac{2(dz)}{s}\left(L + W\right) + \frac{4(dz)^2}{s^2} \qquad \text{(A.2)}$$

where "L" and "W" are the length and width of the pond bottom and "s" is the side slope of the pond. The slope is defined as the change in elevation divided by change in horizontal distance.

The infiltration rate is generally defined as the volume of water that infiltrates per unit time per unit area of the wetted pond bottom:

$$I = \frac{V_{\text{inf}}}{A_{wet}(dt)} \qquad \text{(A.3)}$$

where I = infiltration rate (L/t) and A_{wet} is wetted area of the pond bottom (L^2).

Equations A.1 and A.3 can be combined to give the following expression for infiltration rate during some time interval:

$$I = \frac{V_{\text{inf}}}{A_{wet}(dt)} = \frac{Q_{in}}{A_{wet}} - \frac{A_{surf}(dz)}{A_{wet}(dt)} \qquad \text{(A.4)}$$

During the early portion of the test when the pond is being filled, the flow rate into the pond (Q_{in}) and the change in depth (dz) are both positive values. During the later stage of the test the flow rate is zero and the change in depth is a negative number. In both stages the infiltration rate is a positive value.

Except for ponds with vertical sides, the wetted area of the pond bottom will depend upon the depth of water in the pond. The wetted area (A_{wet}) for a rectangular pond is given by the following expression:

$$A_{wet} = L * W + \left(2L + 2W + \frac{4(dz)}{s} \right) \sqrt{(dz)^2 + \frac{(dz)^2}{s^2}} \quad \text{(A.5)}$$

A.3 An example: Clark County Pond

In November 2000 a flood test was performed at a stormwater infiltration pond in Clark County, shown in Figure A.2. The pond is located in a residential area in the northern part of Vancouver, Washington. No information was available regarding the age of the pond or area of the basin that drains into the pond.

The pond bottom is rectangular with dimensions 75 feet by 25 feet. The side slopes were surveyed to be 0.3 (V:H) and the depth of the pond is approximately six feet. A three-foot high retaining wall constructed with concrete blocks in located on the north side of the infiltration pond. The retaining wall is permeable because of open spaces between the concrete blocks. The water flows into the infiltration pond from a pre-treatment area located on the grassy plateau above the retaining wall. Stormwater discharges to the pretreatment area from two pipes located at opposite ends of the grassy plateau.

The pond bottom and side slopes are covered with patches of moss and grass and two trees are located in opposite diagonal corners of the pond. The side slopes of the pond are covered in patches of grass.

The basic data that were used to estimate infiltration rates at the Clark County site are shown in Table A.1 and in Figure A.3. Table A.1 lists the flow rates, the duration of the test, and the pond geometry. Figure A.3 shows the depth of water as a function of time, measured at 10-minute intervals. The flow rate into the pond during the filling portion of the test was approximately 2000 ft^3/hr (250 gpm) and the filling stage lasted for approximately 2.5 hours. The water level in the pond reached a depth of approximately 21 inches during the filling portion of the test.

Table A.1 - Summary of data for full-scale infiltration tests.

A. Site	B. Inflow Rate (ft^3/hr)	C. Inflow Duration (hours)	D. Total Inflow (ft^3)	E. Draining Duration (hours)	F. Max. Water Depth (inches)	G. Bottom Area (ft^2)	H. Side Slope	I. Total Infiltration (ft^3)
Clark	2060	2.5	4710	67.8	21.5	1860	0.3	4820

The draining phase, which lasted just under 70 hours, can be divided into two parts. During the first part, which lasted until hour 30, the change in water level followed an exponentially shaped curve that became flatter with time. Between hour 30 and the end of the test, the water level followed a linearly shaped curve with a slope that was essentially constant.

The water level data shown in Figure A.3 can be used to estimate the infiltration rate as a function of time using Equation A.4. Figure A.4 shows the infiltration rate averaged over each 30-minute period. The infiltration rate increases with time during the filling portion of the test and reaches a maximum rate of approximately 2.0 in/hr. After the discharge to the pond is stopped, the infiltration rate quickly decreases to approximately 0.25 in/hr. This rate remains roughly constant for the duration of the test.

The total volume of water that was discharged to the infiltration pond during the full-scale test at the Clark County site was approximately 4710 ft^3. All of this water eventually infiltrated into the ground. Figure A.5 shows the fraction of the total inflow that had infiltrated from the pond as a function of time. The total inflow is estimated by summing the infiltration rates shown in Figure A.4 over the duration of the test. This total inflow is given in the last column of Table A.1. It should be noted that the total inflow that is calculated based on estimated infiltration rates (Column "I" in Table A.1) is different from the total inflow that is calculated based on discharge to the pond (Column "D" in Table A.1). These differences are due to uncertainties in pond surface area and wetted pond bottom in Equation A.4.

The fraction of the total inflow that had infiltrated at each time is calculated by summing the infiltration rates for all earlier times and then dividing this sum by the total inflow. For the Clark County site, approximately 20% of the total inflow occurred during the filling stage of the test.

The estimated infiltration rate is nearly constant after approximately 30 hours, as shown in Figure A.4. A constant infiltration rate suggests that the hydraulic gradient that causes flow is also approximately constant during this period. Given that the water level in the pond was decreasing from approximately 10 inches at 30 hours to less than 1 inch at 70 hours, it appears that the water level or water pressure in the pond was not a primary component of the forces causing infiltration.

The rapid increase in infiltration during the filling portion of the test may be caused in part by lateral flow along the sides of the ponds. This is similar to "bank storage" that occurs in stream channels. As the water level in the pond increases, flow is induced horizontally into the banks of the pond. This infiltration is in addition to the infiltration that occurs along the pond bottom. Once the water level in the pond begins to decrease, the horizontal flow is reversed and water drains into the pond along the sides and out of the pond along the bottom. This inflow, which reduces the net infiltration rate, decreases with time.

Figure A.6 compares infiltration rates as a function of water level for the rising and falling limbs of the hydrograph. The graph shows that infiltration rates during the rising limb were significantly larger than the infiltration rates during the falling limb. In general, this is expected because the head gradient is largest for initially dry soils. However, these differences in infiltration rates could also be explained by lateral flow.

When the pond is filling up, flow in both the horizontal and vertical direction allows for more water storage in the sub-surface and thus a higher infiltration rate. When the water level in the pond decreases, the flow from the sides of the pond reverses direction.

The data shown in Figure A.6 can be used to estimate the relative magnitude of the horizontal and vertical flows. As an example, when the depth of water in the pond is18 inches, the infiltration rate during the rising limb is approximately 1.7 in/hr while the infiltration rate during the falling limb is approximately 0.8 in/hr. During the filling phase, the horizontal and vertical flows are both away from the pond, as shown in Figure A.7a. Equation (A.6) describes vertical and horizontal flow from the pond during the filling phase:

$$V + H \cong 1.7 \text{ in/hr.} \qquad \text{(A.6)}$$

where V is vertical flow (L/t) and H is horizontal flow (L/t).

During the draining phase, the horizontal flow is into the pond and the vertical flow is away from the pond, as shown in Figure A.7b. Equation (A.7) describes vertical and horizontal flow from the pond during the draining phase:

$$V - H \cong 0.8 \text{ in/hr.} \qquad \text{(A.7)}$$

Equations A.6 and A.7 can be solved simultaneously if it is assumed that V and H are the same in both equations. This results in a horizontal flow of 0.45 in/hr and a vertical flow of 1.25 in/hr. At the water depth of 18 inches, the horizontal flow accounts for roughly 36 percent of the total infiltrating water. When the water level stops rising, the water infiltrating vertically now has the addition of the total horizontal flow volume of water that is coming back into the pond. This may explain why the net infiltration rate quickly drops as soon as the inflow from the hydrant is shut off.

Figure A.1 – Example pressure transducer set-up.

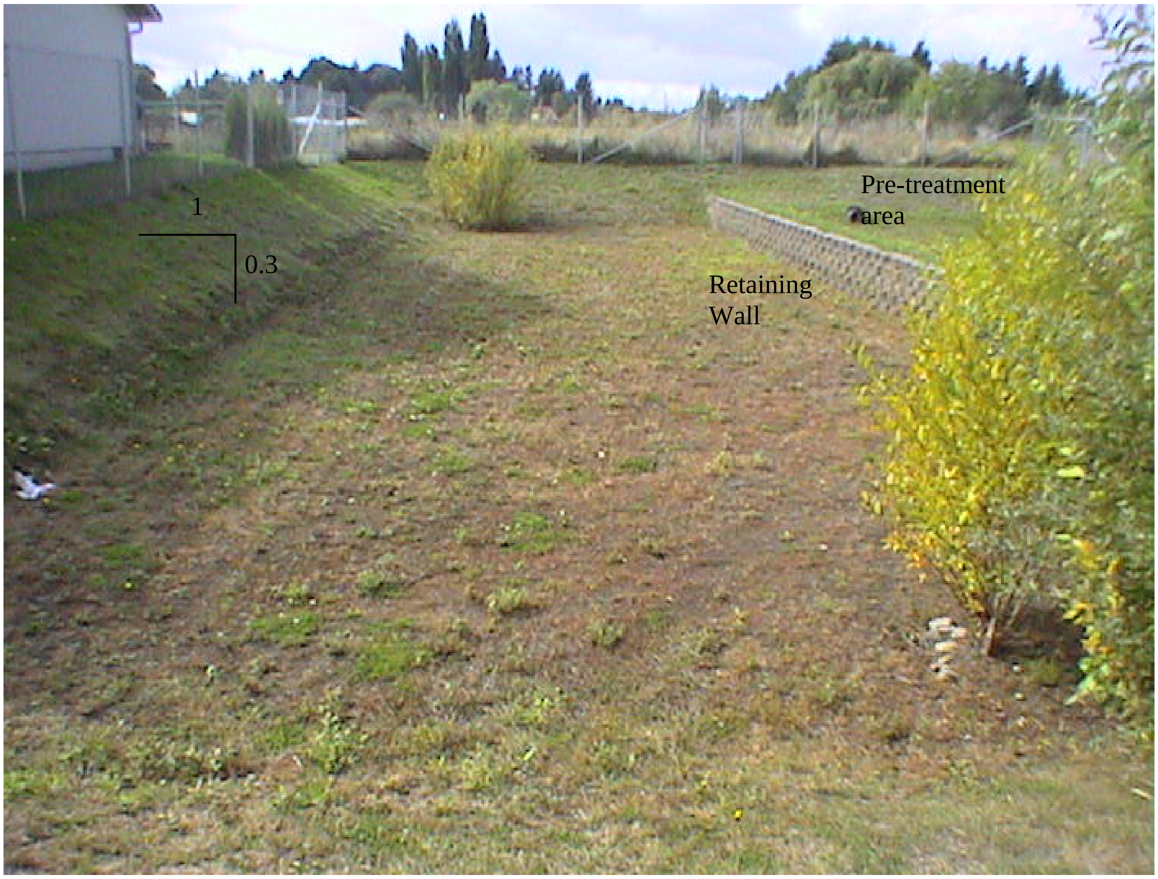

Figure A.2 - Clark County pond, 9616 NE 59th Ave., Vancouver, Washington, 98686

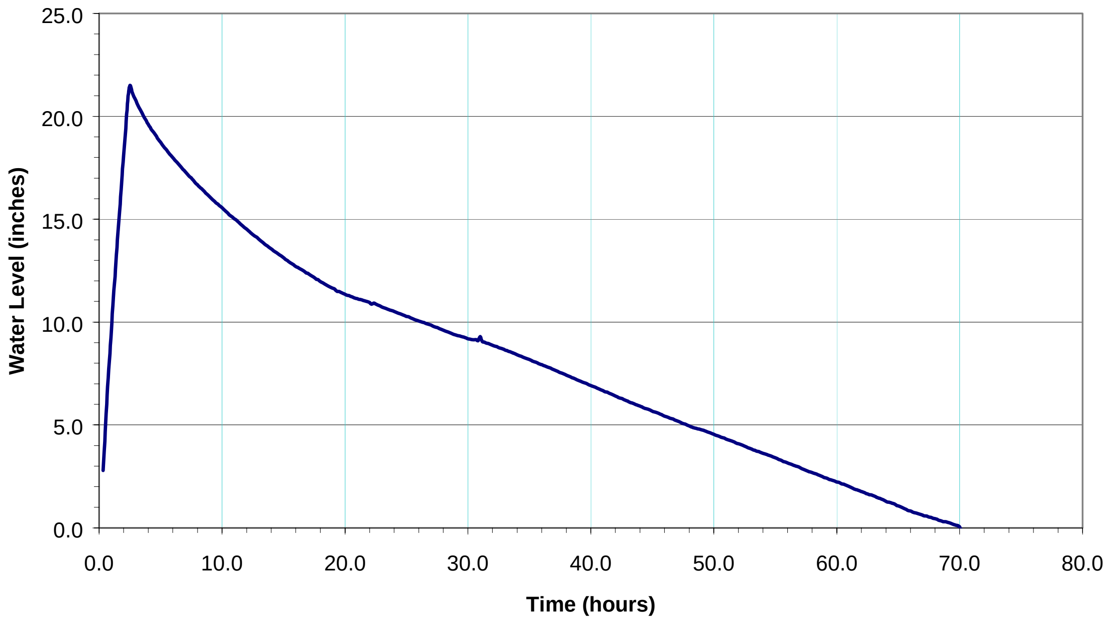

Figure A.3 - Water levels during the full-scale test at the Clark County Pond.

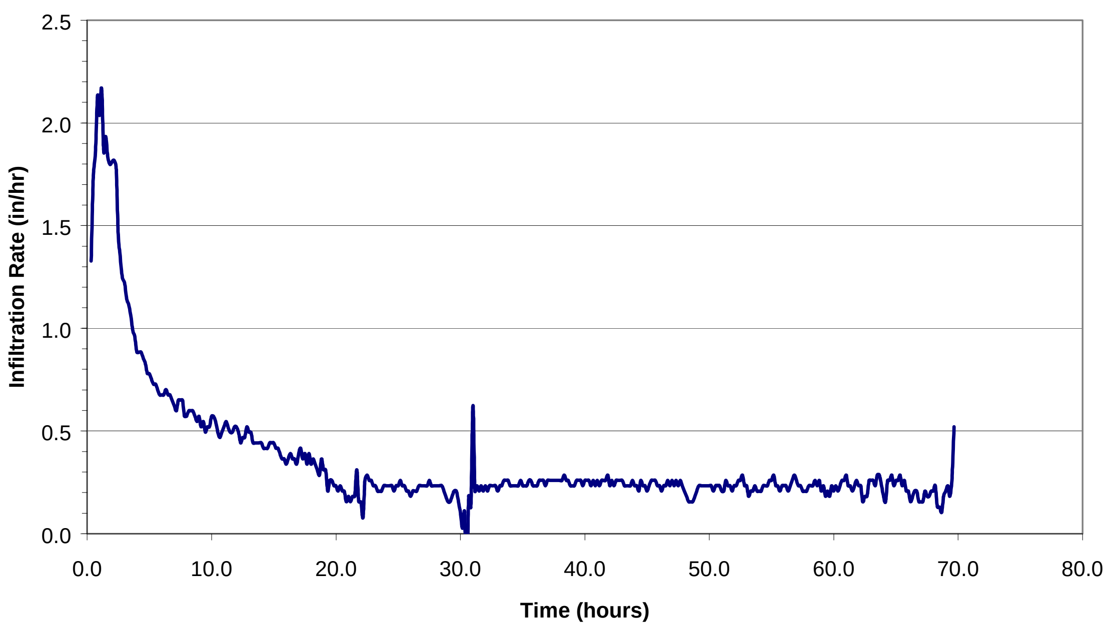

Figure A.4 - Infiltration rate for Clark County averaged over 30-minute intervals.

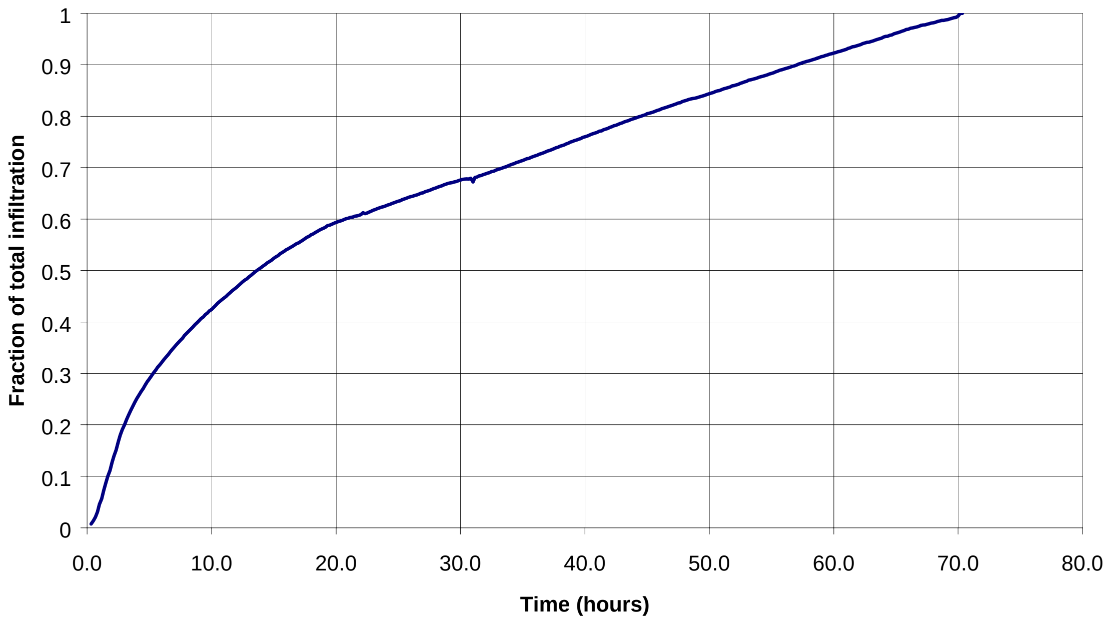

Figure A.5 - Fraction of total inflow that had infiltrated as a function of time at the Clark County Pond.

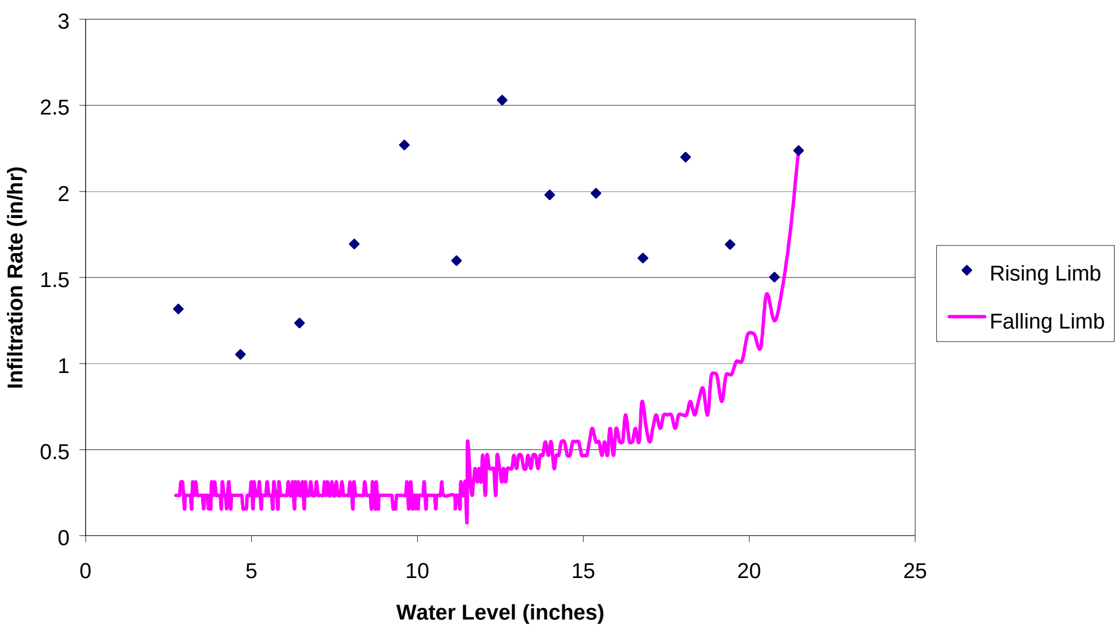

Figure A.6- Infiltration rate versus water level for Clark County Pond.

a) Filling - Equation (A.6)　　　b) Draining - Equation (A.7)

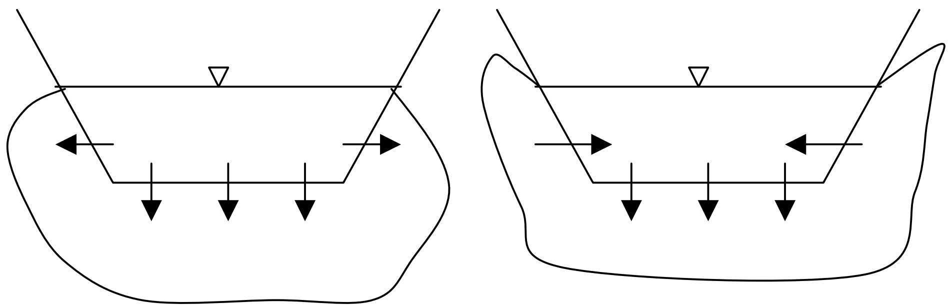

Figure A.7 - Horizontal and vertical infiltration during filling and draining stages.

APPENDIX B: EXAMPLE CALCULATIONS USING THE SUGGESTED DESIGN APPROACH

This appendix describes examples calculations used to estimate infiltration rates for ponds. The focus of the calculations is on estimating infiltration rates using the approaches and equations described in Sections 2.4 through 2.7. The examples are based on field data collected at four existing facilities described in the Thurston County study (Wiltsie, 1998; Massmann et al., 2003). Infiltration rates are estimated for the following sites: Airdustrial, Lacy Lid, Sweetbriar, and Woodward Glen. These four sites were selected because of the availability of data describing groundwater levels. Descriptions of the four sites are included in Table B.1.

B.1 Estimate saturated hydraulic conductivity from soil information

Saturated hydraulic conductivity values will be estimated in the examples using soil texture information, as described in Section 2.4.1. Table B.2 includes soil texture information that was collected at the four sites. The grain-size curves that used to develop this data are included in Appendix C in Massmann et al. (2003).

The first column in Table B.2 describes the number of horizontal locations at which soil samples were collected at each facility. The second column describes the number of layers that were encountered at each horizontal location and the third column gives the thickness of each of these layers. Columns E through H in Table B.2 gives soil texture information. This information includes D_{10}, D_{60}, and D_{90} grain size diameters and the fraction of fine-grained material, f_{fines}, for each horizontal location and for each layer.

The soil texture information can be used to estimate the saturated hydraulic conductivity value using the log-regression equation (Equation 2.2):

$$\log_{10}(K_{sat}) = -1.57 + 1.90D_{10} + 0.015D_{60} - 0.013D_{90} - 2.08\mathrm{f}_{\mathrm{fines}} \qquad (2.2)$$

Column I in Table B.2 gives saturated hydraulic conductivity values in cm/s for each location and layer calculated using Equation 2.2. These values represent "point" measurements for the hydraulic conductivity of each layer at each location. The values for each layer can be combined using Equation 2.4 to give an equivalent hydraulic conductivity for each location:

$$K_{equiv} = \frac{d}{\sum \frac{d_i}{K_i}} \qquad (2.4)$$

These equivalent hydraulic conductivity values are given in Column J in Table B.2.

Finally, the equivalent hydraulic conductivity values for each location can be averaged to obtain a single estimate of saturated hydraulic conductivity value for each facility. This averaging is described in Column K in Table B.2. The average hydraulic conductivity values in units of ft/day are included in Column L.

B.2 Estimate the hydraulic gradient

The hydraulic gradient at each site can be estimated using Equation 2.6:

$$\text{gradient} = \text{i} \approx \frac{D_{wt} + D_{pond}}{138.62(K^{0.1})} CF_{size} \qquad (2.6)$$

where K is saturated hydraulic conductivity in feet/day, D_{wt} is the depth in feet from the base of the infiltration facility to the water table or to the first low-permeability layer, and D_{pond} is the depth of water in the pond, in feet.

The correction for pond size, CF_{size}, is given by Equation 2.7:

$$CF_{size} = 0.73(A_{pond})^{-0.76} \qquad (2.7)$$

where

CF_{size} = correction factor for size of the pond

A_{pond} = area of the pond bottom in acres

Equation 2.7 was developed for ponds with bottom areas between 0.6 and 6 acres in size. For small ponds (ponds with area equal to 2/3 acre), the correction factor is equal to 1.0. All of the example facilities were smaller than 2/3 acre so that correction factor for pond size is 1.0 in all cases.

Table B.3 summarizes gradient calculations. The depths to the water table at the four sites, D_{wt}, and the depth of water in the pond D_{pond}, were estimated based on data that were collected using pressure transducers at each site. These data are described in Wiltsie et al. (1998). The hydraulic conductivity values in column E were estimated using the log regression equation, as described in the previous section. Column F gives estimated gradient for each location.

B.3 Estimate the uncorrected infiltration rate

The infiltration rates are estimated by multiplying the saturated hydraulic conductivity with the hydraulic gradient, as described by Equation 2.9:

$$f = K\left(\frac{dh}{dz}\right) = Ki \qquad (2.9)$$

Table 8.14 summarizes the calculated infiltration rates for the example sites.

B.4 Apply correction factors for pond geometry and for biofouling and siltation

The infiltration rates calculated in Table B.4 do not include corrections for pond geometry or for biofouling and siltation caused by poor maintenance and/or insufficient pretreatment. The correction factor for pond geometry is given by Equation 2.10:

$$CF_{aspect} = 0.02A_{ratio} + 0.98 \qquad (2.10)$$

Table B.5 includes aspect ratios for the four example ponds. The calculated correction factor for pond geometry is also included in Table B.5.

Correction factors for biofouling and siltation are included in Table 2.7. These correction factors describe both biofouling and poor maintenance. The report by Wiltsie (1998) includes descriptions of pond conditions, including biofouling and maintenance practices. The correction factors in Table B.6 were estimated based on these descriptions. Two estimates of infiltration rate are included for the Airdustrial site. The first estimate assumes poor maintenance with a correction factor for siltation and biofouling equal to 0.3. The second estimate assumes a maintained facility with a correction factor equal to 1.0. These two estimates were developed to allow comparisons with observed infiltration rates from tests that were conducted both before and after maintenance activities at the actual field site.

The infiltration rates calculated in Table B.5 are corrected using the correction factors for aspect ratio and for maintenance and biofouling in Table B.6. These corrected infiltration rates are calculated using Equation (2.11):

$$f_{corr} = (CF_{silt/bio})(CF_{aspect})f = (CF_{silt/bio})(CF_{aspect})Ki \qquad (2.11)$$

B.5 Comparison of observed and calculated infiltration rates

Table B.7 provides a comparison of the measured infiltration rates with the estimates that were developed using the suggested design approach. The rates are reasonably similar, in part because the correction factors used for siltation and biofouling were originally developed based on observations at these ponds. Table B.7 also includes estimates used the WDOE approach based on the D_{10} grain size diameters. The values calculated using the equations suggested in the current study incorporate more site-specific characteristics, including pond size, depth to groundwater, and depth of water in the pond. These additional characteristics allow more variability in the estimated infiltration rates, as shown in Table B.7. The WDOE recommended rates based on USDA soil textural classification and D_{10} measurements (reproduced in Tables 2.8 and 2.9) do not explicitly consider hydraulic gradient. The WDOE rates are based on observations from sites in Western Washington with shallow groundwater tables and with absent or inconsistent pretreatment and maintenance practices. As expected, these rates are reasonably similar to the rates estimated using the approach developed in the current study for these particular sites in Western Washington. The two approaches would give significantly different results for facilities with deeper water tables and with better pretreatment and maintenance activities.

Table B.1 – Description of ponds used in example calculations

Site Name	Site Address	Pond Age (years)	Pond Bottom Surface Area (ft^2)	Pond Volume (ft^3)	Pond Geometry
Airdustrial	Bonniewood Dr SW and 70th Ave Tumwater, WA	7	6,400	34,000	Rectangular
Lacey Lid	Yelm Hwy and Corporate Ctr Lacey, WA	10	17,100	248,276	Rectangular
Margaret McKenny	Morse-Merryman Rd SE and Quentin St.; Lacey, WA	10	6,720	72,352	Rectangular
Sweetbriar	Boulevard Rd and 45th Av. SE Lacey, WA	8	15,000	92,123	Triangular
Woodard Glen	Lister Rd NE and Cherry Blossom Olympia, WA	21	2,000	8,700	Small trapezoidal

Table B.2 - Estimates of saturated hydraulic conductivity developed using the log-regression relationship

A	B	C	D	E	F	G	H	I	J	K
Site	**Location**	**Layer**	**Thickness (in.)**	**d_{10} (mm)**	**d_{60} (mm)**	**d_{90} (mm)**	**fines**	**K_s (cm/s)**	**K_{equiv} (cm/s)**	**K_{equiv} (ft/day)**
Airdustrial	1	1	67"	0.2	0.3	0.4	0.02	5.8E-02	5.8E-02	
	2	1	48"	0.13	0.31	0.7	0.03	4.0E-02	4.0E-02	
								Average K_{equiv}	**4.9E-02**	**140**
Lacey Lid	1	1	69”	0.11	0.26	0.4	0.03	3.7E-02	3.7E-02	
	2	1	50”	0.16	0.32	0.7	0	5.3E-02	5.3E-02	
								Average K_{equiv}	**4.5E-02**	**128**
Margaret McKenny	1	1	22"	0.34	12	31	0.02	6.34E-02		
	1	2	20"	0.23	1	10	0	5.55E-02		
	1	3	12"	0.11	0.8	11	0.06	2.37E-02	4.4E-02	
								Average K_{equiv}	**4.4E-02**	**126**
Sweetbriar	1	1	36”	0.16	0.28	0.4	0	5.34E-02		
	1	2	9”	0.11	0.28	0.41	0.03	3.72E-02		
	1	3	24”	0.18	0.3	0.5	0	5.82E-02	5.2E-02	
	2	1	36”	0.18	0.29	0.4	0.02	5.30E-02		
	2	2	30”	0.21	0.52	0.94	0	6.60E-02	5.8E-02	
								Average K_{equiv}	**5.5E-02**	**156**
Woodard Glen	1	1	15"	0.55	9	11.7	0	2.84E-01		
	1	2	28"	0.2	0.55	1.5	0.01	5.92E-02	8.2E-02	
	2	1	36"	0.18	0.28	0.4	0.01	5.56E-02		
	2	2	6"	0.2	40	100	0	1.24E-02	3.7E-02	
								Average K_{equiv}	**5.9E-02**	**168**

Table B.3– Estimates of hydraulic gradient for the example sites

A	B	C	D	E	F
Facility	**A_{pond} (acres)**	**D_{wt} (feet)**	**D_{pond} (feet)**	**K (ft/day)**	**Calculated gradient**
Airdustrial	0.15	3.0	1.0	140	0.018
Lacey Lid	0.52	2.5	0.4	128	0.013
Margaret McKenny	0.15	3.5	1.3	126	0.021
Sweetbriar	0.34	3.2	1.7	156	0.021
Wood Glen	0.05	3.2	0.6	168	0.016

Table B.4 – Estimates of uncorrected infiltration rates for the example sites

A	B	C	D	E
Facility	**K (ft/day)**	**K (in/hr)**	**Calculated gradient**	**Calculated infiltration rate (in/hr)**
Airdustrial	140	70	0.018	1.23
Lacey Lid	128	64	0.013	0.83
Margaret McKenny	126	63	0.021	1.33
Sweetbriar	156	78	0.021	1.64
Wood Glen	168	84	0.016	1.38

Table B.5 – Correction factors for aspect ratios

Facility	Aspect Ratio	CF_{aspect}
Airdustrial	1.0	1.0
Lacey Lid	18.0	1.3
Margaret McKenny	5.7	1.1
Sweetbriar	1.5	1.0
Woodward Glen	1.4	1.0

Table B.6 – Correction factors for siltation and biofouling

Facility	Descriptions Regarding Siltation and Biofouling	CF_{aspect}	$CF_{silt/bio}$	Infiltration (in/hr)	
				Uncorrected	**Corrected**
Airdustrial	Not maintained	1.0	0.3	1.23	0.37
Airdustrial	Maintained	1.0	1	1.23	1.23
Lacey Lid	Silt fouling	1.3	0.5	0.83	0.56
Margaret McKenny	Partially maintained	1.1	0.9	1.33	1.31
Sweetbriar	Not maintained	1.0	0.3	1.64	0.50
Woodward Glen	Maintained	1.0	1	1.38	1.39

Table B.7. Comparison of measured infiltration rates with the estimated rates

A	B	C	D	E
Facility	**Estimated infiltration (in/hr)**	**Observed infiltration rate (in/hr)**	**Rate estimated from WDOE[a] (in/hr)**	**d_{10} (mm)**
Airdustrial (pre-maintenance)	0.37	0.3	2	0.09
Airdustrial (post-maintenance)	1.23	1.7	2	0.09
Lacey Lid	0.56	0.3	2	0.12
Margaret McKenny	1.31	2.0	3.5	0.25
Sweetbriar	0.50	0.4	0.8	0.065
Woodward Glen	1.39	2.3	0.2	0.1

[a]Rate based on D_{10} using Table 3.8 in Vol. III and Table 7.2 in Vol. V, WDOE, 2001