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INFILTRATION GUIDE

Near-Term Action (NTA) 2018-0827: Flexible Infiltration Test Methods for Evaluating Infiltration Feasibility

Prepared for: **City of Tacoma**

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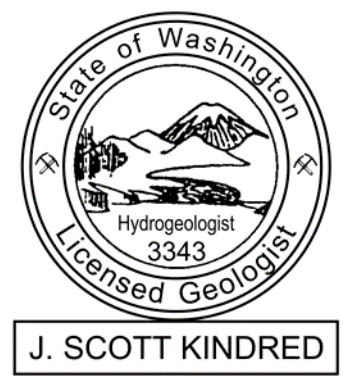

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Abstract

This infiltration guide (Guide) is the culmination of a larger study to evaluate different infiltration test methods and provide an expanded toolbox for evaluating stormwater infiltration feasibility and estimating infiltration capacity for different stormwater infiltration facilities. The purpose of the Guide is to provide standardized procedures and requirements for infiltration assessments that are consistent with Washington State's Stormwater Management Manual for Western Washington. Most of the procedures and requirements provided in this Guide are based on detailed technical analysis summarized by Kindred (2022).

The Guide provides guidelines for a multi-step process that includes a feasibility assessment, field infiltration testing procedures, calculation of bulk hydraulic conductivity (K_b) and design hydraulic conductivity (K_d), using K_d to estimate infiltration capacity for both horizontal infiltration facilities and drywells, and general procedures for conducting groundwater mounding assessments.

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1 General

1.1 Limitation of Current Infiltration Methodology

The current Washington State's Stormwater Management Manual for Western Washington (SWMMWW, WSDOE, 2019) provides three methods for estimating the measured infiltration rate: the large-scale pilot infiltration test (PIT), the small-scale PIT, and the grain size method (for normally consolidated soils only). Unfortunately, the SWMMWW sometimes incorrectly treats the term saturated hydraulic conductivity (K_{sat}) as equivalent to infiltration rate (I). This is inaccurate, however, since K_{sat} is a soil property and $I = K_{sat} \times i$, where i is the hydraulic gradient.

The PIT method maintains a steady ponding depth for at least 6 hours in a test pit with an area of at least 12 ft^2 for the small-scale test and 100 ft^2 for the large-scale test. The measured K_{sat} (should be I) is calculated by dividing the flow rate at the end of the test by the area of the test pit. The grain size method is based on the Massman (2003) equation and relies on a variety of grain size parameters to estimate K_{sat} for each of the layers below the proposed facility. It then uses the harmonic mean to estimate the effective K_{sat}.

The SWMMWW provides two methods for calculating the design K_{sat} from the measured K_{sat}: the simplified approach and the detailed approach. The simplified approach calculates the design K_{sat} by multiplying the measured K_{sat} by a series of correction factors (site variability, test method, and degree of influent control). The detailed approach (Massman 2003) is based on the depth to groundwater, the depth of ponding in the facility, and the size of the pond. To further confuse the issue, the detailed approach does multiply K_{sat} by i to provide an estimate for I and the simplified approach does not include this step. It should be noted that the detailed approach is very conservative and results in low design infiltration rates.

Although the SWMMWW provides design K_{sat} using the simplified method, the hydrologic models (e.g., the Western Washington Hydrologic Model) used to simulate the performance of proposed infiltration facilities generally use I. It should be noted that these models only account for vertical flow out of the bottom of the facility and do not account for variations in I due to the size of the facility or the ponding depth.

The current methodologies for determining design infiltration rate in the SWMMWW (described as "long-term design native soil infiltration rate" in the manual) have significant shortcomings, summarized below:

- The SWMMWW improperly uses the term K_{sat} when it should be using I.
- PITs are difficult to conduct in developed areas with limited access and subsurface infrastructure.
- No methods are provided for estimating the capacity of deep drywells which are dominated by horizontal flow out the sides of the well.
- The PIT method provides an infiltration rate without accounting for the size of the facility or the depth of ponding during the test. Numerical modeling has shown that infiltration rate can vary significantly as the depth of ponding and the area of the facility changes.
- The grainsize method for calculating hydraulic conductivity is not suitable for glacially consolidated soils and only provides a very general estimate of hydraulic conductivity. Furthermore, this method assumes one-dimensional plug flow and does not account for horizontal flow due to layering.
- The correction factors used by the simplified approach to calculate the design infiltration rate are not specified in sufficient detail and do not address all the factors that may affect the performance of a full-scale facility.
- The detailed approach has numerous issues, the primary one that it assumes plug flow through a homogeneous isotropic soil and does not accurately represent flow dynamics from an infiltration facility.

- The infiltration rate based on testing in a small test pit will significantly over-estimate the infiltration rate for a full-size infiltration facility.
- The hydrologic models used to simulate the performance of infiltration facilities assume that infiltration rate is a soil property and do not account for differences in facility area or ponding depth.

The methods provided in this Guide will address all of these issues.

1.2 Purpose

The purpose of this infiltration guide (Guide) is to provide procedures and requirements for infiltration assessments that achieve the following:

1. Deliver infiltration facilities that are appropriately sized and designed to achieve water quality and flow control objectives consistent with Washington State's Stormwater Management Manual for Western Washington (SWMMWW).
2. Address the shortcoming of the current infiltration design rate determination methodologies provided in the SWMMWW.
3. Provide standardized procedures and guidelines that are simple and easy to implement.
4. Ensure that the level of site characterization, testing, and analysis is appropriate for the size of the project and potential risk.
5. Simplify the review process for the stormwater permitting agency.

The scope of this Guide is limited to infiltration testing methods and associated analysis to support the design and sizing of infiltration facilities. It does not specifically address the infeasibility criteria provided in Section V-5 of the SWMMWW for infiltration BMPs nor the site suitability criteria provided in Section V-5.6 of the SWMMWW (i.e., setbacks, groundwater protection areas, high vehicle traffic areas, treatment criteria, depth to bedrock, water table or a low permeability layer, seepage analysis and control, and roadway deicers).

1.3 Uncertainty

Subsurface conditions are highly variable and the performance of infiltration facilities can vary significantly with small changes in location and depth of the facility (Kindred, 2022, Volume IV). This uncertainty can be reduced with greater levels of explorations and testing. However, site characterization and infiltration analysis are expensive, and the level of effort should be commensurate with the size and the potential risk of the project.

Given the variability of subsurface conditions and uncertainty regarding the performance of infiltration facilities, some infiltration facilities will outperform design expectations and some will underperform design expectations. For large infiltration facilities, the adverse impacts associated with underperformance can be significant and a higher level of site characterization and infiltration analysis, along with a more conservative design infiltration rate, is warranted. For small sites, the adverse impacts associated with underperformance are less significant and a lower level of site characterization and analysis, along with a less conservative design infiltration rate, are appropriate.

1.4 Risks Associated with Stormwater Infiltration

Stormwater infiltration provides many benefits compared with standard stormwater management practices that utilize detention and conveyance. Flow through the subsurface provides an additional level of water quality treatment above and beyond typical treatment approaches. In addition, stormwater infiltration provides additional groundwater recharge that is available for groundwater extraction and baseflow in streams during the dry season. However, increasing groundwater recharge may increase the potential for landslides in areas with steep slopes, cause surface seepage and flooding that impacts existing development, or increase migration of subsurface contamination. Stormwater infiltration is not appropriate in some areas due to the potential for adverse impacts. Infiltration assessments should consider the potential for adverse impacts and stormwater infiltration should not be allowed if there is a reasonable potential for adverse impacts.

Although it is known that stormwater infiltration affects the potential for landslides and flooding, characterizing and quantifying these risks is fraught with uncertainty and can be expensive. This Guide does not specify how to address the risk of adverse impacts. Obviously, larger infiltration facilities are more likely to result in adverse impacts compared with smaller infiltration facilities. Although small, widely-spaced infiltration facilities may have an insignificant impact on groundwater elevations and flow, the impacts may become significant as the density of small infiltration facilities increases. Individual landowners cannot be expected to evaluate the cumulative impacts of infiltration facilities on neighboring properties they do not own. The stormwater regulations should identify the entities responsible for identifying and assessing the potential risks associated with cumulative impacts of multiple small projects and developing limitations on stormwater infiltration based on these assessments.

This document provides the minimum investigation requirements for infiltration assessments. This information does not preclude the use of professional judgment to evaluate and manage risk associated with design, construction, and operation of infiltration facilities. Recommendations that deviate from this document shall be contained in a stamped and signed letter from a qualified professional (as defined in Section 1.5) and must provide rationale and specific data supporting their professional judgment.

1.5 Qualified Professionals

This Guide is designed to provide clear and concise procedures and requirements that are relatively straight-forward and do not require a great deal of specialized training. Never-the-less, it is important that the work be conducted by, or under the supervision of, qualified professionals. With any project, there may be special circumstances and/or unusual conditions that warrant changes to the procedures and/or requirements and the judgement of the qualified professional is critical to identify and address these circumstances and conditions.

Most of the procedures and assessments outlined in this Guide should be conducted under the supervision of a qualified infiltration professional, defined as a professional engineer or licensed geologist with at least five years of experience conducting infiltration assessments (experience must include a minimum of ten sites). There are two exceptions: landslide hazard assessments and groundwater mounding assessments. Any assessments of landslide hazards on or down-gradient of the site should be conducted by a qualified geotechnical professional, defined as a professional engineer or licensed engineering geologist with at least five years of experience conducting slope stability assessments (experience must include a minimum of ten sites). Groundwater mounding analyses should be performed by a qualified hydrogeologist, defined as a licensed hydrogeologist with at least five years of experience in groundwater assessments, including groundwater modeling (experience must include a minimum of ten sites). The qualified professionals are responsible for tracking their experience and providing a list of project experience to the permitting agency when requested.

2 Feasibility Assessment

As discussed previously, stormwater infiltration is not feasible at every site. At some sites, infiltration is not feasible because the groundwater table is too high or the soils have a relatively low permeability. At other sites, infiltration is not recommended because it may increase the frequency and severity of landslides, the potential for flooding or nuisance water in below-grade structures (such as basements), or increase the potential for surface flooding. As discussed below, the feasibility assessment includes both a desktop assessment and, when necessary, a field assessment.

The infiltration feasibility assessment should be conducted early in the site design process. Close coordination between the qualified infiltration professional and the site design engineers will improve the efficiency of the design process and site characterization. In many cases, the scope of the standard geotechnical site assessment can be modified and/or expanded to address infiltration feasibility. A desktop infiltration assessment and field feasibility screening-level assessment should be conducted before conducting infiltration testing or groundwater mounding assessments.

2.1 Desktop Assessment

Before conducting a field infiltration assessment, the qualified infiltration professional should review readily available existing information, including geologic maps, topographic maps, proximity to landslide hazards and steep slopes, proximity to streams and wetlands, infiltration feasibility maps, existing well and borehole logs, and previous explorations near the site. This assessment should address the infeasibility criteria in Section V-6 of the SWMMWW for infiltration best management practices (BMPs) and any applicable site suitability criterion, such as those provided in Section V-5.6 of the SWMMWW or other criteria specified by the local agency. There may be other risk factors not addressed by the SWMMWW or the local agency and this assessment should rely on the judgement of the qualified infiltration professional to identify and address these risk factors. These desktop assessments should be quick and provide sufficient information to determine if a field feasibility assessment is necessary and help focus the scope of the field feasibility assessment. A field infiltration assessment and infiltration testing are not required if infiltration is deemed infeasible based on documentation developed during the desktop assessment.

Based on the desktop assessment and site development plans, infiltration feasibility can be evaluated and a preliminary field assessment approach can be developed to support stormwater infiltration design. In general, smaller sites (less than 10,000 sf of impervious surface) need only evaluate the feasibility of shallow infiltration (within the upper 10 ft of final grade). Larger sites should consider the feasibility of both shallow and deep infiltration (deeper than 10 ft below final grade).

2.2 Field Feasibility Screening Assessment

If results from the desktop assessment (Section 2.1) indicate that infiltration is potentially feasible, a field feasibility assessment should then be conducted to further assess site infiltration feasibility. This screening-level assessment does not include all the testing and subsurface investigation requirements for infiltration design, which are outlined in Section 3. This screening-level field assessment should include the following:

1. Looking for seepage, wetlands, or surface water that may reflect groundwater elevations,
2. Identifying nearby structures and/or steep slopes that might be impacted by stormwater infiltration,
3. Evaluating site access and water availability for infiltration testing,
4. When feasible, conducting subsurface explorations, such as hand-auger borings. vactor explorations, or excavated test pits, to observe shallow subsurface conditions.
5. Evaluate the minimum separation from groundwater and perching layers, as summarized in Section 2.3 and 2.4.

The field feasibility assessment may determine that shallow infiltration at the site is infeasible. Conditions that may eliminate infiltration as an option may include, for example, groundwater or low permeability soils near the ground surface, landslide hazards on or near the site, or existing seepage downgradient of the site that is impacting existing development.

Some sites that are unsuitable for shallow infiltration may be suitable for deep infiltration. It can be expensive to conduct the deep explorations necessary to evaluate the feasibility of deep infiltration and deep infiltration is generally not cost effective for small developments (less than 10,000 sf of impervious surface). However, if there is no off-site point of discharge, deep infiltration may be the best option for a small site. The feasibility of deep infiltration may be evaluated using existing information, such as well-defined geologic conditions and/or nearby boring logs. If the feasibility cannot be assessed using existing information, one or more deep boreholes should be drilled to determine if there is a suitable infiltration receptor horizon with sufficient separation from groundwater.

As outlined in Section 2.1, there are a number of infeasibility and site suitability criteria that may eliminate infiltration as an option, even when infiltration is feasible. Infiltration testing is not required if infiltration is deemed infeasible based on documentation developed during the field feasibility assessment

2.3 Minimum Separation from Groundwater/Perching Layer

The 2019 Stormwater Management Manual for Western Washington (SWMMWW, WSDOE 2019) requires a minimum separation between the bottom of an infiltration facility and groundwater or a low permeability perching layer. This separation varies depending on the type of infiltration facility.

As described in Section V-4 BMP T5.10A of the SWMMWW (Downspout Full Infiltration), infiltration trenches or drywells intended for full infiltration of roof runoff must have at least 1 ft of clearance between the bottom of the infiltration facility and the seasonal high groundwater table.

As provided in Section V-5.6 of the SWMMWW, Site Suitability Criterion 5 (Depth to Bedrock, Water Table, or Impermeable Layer) requires that infiltration basins and trenches shall be ≥ 5 ft above the seasonal high-water mark, bedrock (or hardpan) or other low permeability layer. A separation down to 3 ft may be considered if the ground water mounding analysis, volumetric receptor capacity, and the design of the overflow and/or bypass structures are judged by the site professional to be adequate to prevent overtopping and meet the other site suitability criteria specified in Section V-5.6.

As described in Section V-5-BMP T7.30 of the SWMMWW (Bioretention) the minimum vertical separation between the base of the bioretention facility and seasonal high-water table, bedrock, or other impervious layer is either 1 ft (if the facility will serve a drainage area that is less than 5,000 ft^2 of pollution-generating impervious surface, 10,000 ft^2 of impervious surface, and ¾ acres of pervious surface) or 3 ft, for larger drainage areas.

Section V-5-BMP T5.15 of the SWMMWW (Permeable Pavements) specifies at least one foot between the bottom of the permeable pavement facility and seasonal high ground water. In addition, permeable pavement is considered infeasible if there is a low permeability layer that would create saturated conditions within one foot of the bottom of the permeable pavement. The bottom of the permeable pavement facility is the bottom of the lowest layer that has been designed to be part of the BMP, such as the lowest gravel base course or a sand layer used for treatment below the permeable pavement.

Different groundwater separation requirements are specified for underground injection control (UIC) wells. UIC wells are any infiltration facility that is deeper than its largest horizontal dimension. UIC wells can be either drilled or dug and are referred to as drywells throughout this Guide. As outlined in Section I-4.10 (Siting and Design of New UIC Wells) of the SWMMWW, any UIC well authorized using the presumptive approach requires at least 5 ft of separation between the bottom of the facility and the groundwater table, bedrock, hardpan, or other low-permeability layer. This separation may be reduced to 3 ft if the demonstrative approach demonstrates that this will meet the non-endangerment standard. Requirements of the demonstrative approach are provided in Section I-4.9 of the SWMMWW. Unfortunately, the UIC requirements for groundwater separation are different than the requirements for downspout full infiltration drywells in Section V-4 of the SWMMWW. This inconsistency should be addressed in the next revision of the SWMMWW.

As outlined in Section I-4.15 (Deep UIC Wells) of the SWMMWW, any drywell that penetrates an upper confining layer and discharges into the underlying vadose zone must have at least 15 ft of separation between the bottom of the drywell and the seasonal high groundwater table. The SWMMWW does not define "upper confining layer", although the common understanding is glacial till or other low-permeability material over advance outwash. This requirement does not refer to separation from a low permeability perching layer.

2.4 Definitions for Seasonal High Groundwater and Perching Layers

As discussed in Section 2.3, the SWMMWW specifies minimum separation between the bottom of an infiltration facility and the seasonal high groundwater table or a low permeability perching layer. Definitions for each of these conditions are provided below.

2.4.1 Seasonal High Groundwater

Groundwater levels can vary from year to year and previous guidance for determining seasonal high groundwater has been vague. For the purposes of this Guide, seasonal high groundwater is defined as the highest saturated zone below the target infiltration interval. For shallow infiltration facilities, this is the saturated zone closest to the ground surface. For deep drywells that have a surface seal, this is the uppermost saturated zone below the filter pack interval.

Groundwater levels can be based on direct groundwater measurements in a groundwater monitoring well screened within 10 ft of the top of the aquifer, seepage observed in a drilled or dug exploration, or signs of orange soil staining indicative of periodic saturation in response to precipitation events. In some cases, there is existing information (e.g., well logs or geologic maps) that can be used to determine that the seasonal high groundwater table is well below the bottom of the proposed facility and no additional subsurface information is required.

The seasonal high groundwater table is based on groundwater conditions before operation of the proposed infiltration facility. The concentrated recharge associated with an infiltration facility will generally create a groundwater mound that can potentially rise up to the bottom of the infiltration facility during storm events. The groundwater mound associated with site development will be addressed using a groundwater mounding correction factor or conducting a groundwater mounding assessment.

In cases where the seasonal high groundwater table is unknown or potentially within the required separation, the level of effort to characterize seasonal high groundwater depends on the size of the site. For small sites (i.e., less than 10,000 sf of impervious surface) one-time observations during site characterization and/or installation of the infiltration test facility are sufficient to document the seasonal high groundwater table. For larger sites it may be necessary to install one or more groundwater monitoring wells and measure water levels during the period of highest groundwater elevations. The period of highest groundwater elevation can vary depending on precipitation, irrigation patterns, groundwater pumping patterns, and the depth to groundwater. Generally, a qualified hydrogeologist can estimate the period of highest groundwater elevations for a specific site.

2.4.2 Perching Layer

Separation between an infiltration facility and a low permeability perching layer is important because the concentrated recharge associated with an infiltration facility may create a groundwater mound on a low permeability layer that did not create a year-round saturated zone under background recharge conditions. The presence of these layers can be detected based on grainsize information and/or evidence of periodic groundwater perching, including the presence of wet layers and/or orange staining. Orange staining generally indicates oxidation of iron-based minerals associated with periodic saturated conditions.

The rate of flux through a low permeability layer and the potential for groundwater mounding is a function of the saturated hydraulic conductivity (K_s) of the layer and the thickness of the layer. Low permeability layers that are less than 6-in. thick are less likely to cause groundwater mounding than layers that are multiple feet thick. The qualified infiltration professional should determine if the layer is likely to cause a significant groundwater mound based on a combination of considerations, including the evidence for perching, evidence for groundwater mounding during infiltration testing (see Section 5.3), the grainsize characteristics, the lateral extent of the layer, and the thickness of the layer.

2.5 Reporting

A summary of the desktop assessment and the field feasibility screening assessment shall be provided in the Preliminary Infiltration Feasibility Report. This report shall include a description of the work completed, a description of the relevant surface and subsurface characteristics of the site, an evaluation of any applicable infeasibility criteria, and preliminary conclusions regarding the feasibility of infiltration at the site.

3 Field Infiltration Testing Procedures

This section outlines site characterization and infiltration testing procedures to support design of infiltration facilities. The purpose of infiltration testing is to estimate the bulk hydraulic conductivity (K_b) of the soil horizon targeted by the proposed infiltration facility. Using methods provided in Sections 4 - 6 of this Guide, K_b can be used to estimate the capacity of the proposed infiltration facility. Information provided by the desktop assessment and field feasibility assessment, combined with the preliminary stormwater site plan, provide the basis for the infiltration test plan.

3.1 Test Design for Horizontal Infiltration Facilities

A horizontal infiltration facility is defined as an infiltration facility with an equivalent radius (r_e) greater than the maximum ponding depth of the facility (H_{max}) and infiltration capacity is dominated by vertical flow out of the bottom of the facility. Generally, infiltration ponds, bioretention facilities, and permeable pavements are horizontal infiltration facilities. Equivalent radius is defined as $r_e = \sqrt{AREA_{\max}/\pi}$, where $AREA_{max}$ (ft^2) is the surface area of the pond in the facility when the ponding depth (H) = H_{max}.

Ideally, the location and elevation of the proposed infiltration facility are known in advance of infiltration testing. One test per horizontal infiltration facility is sufficient if the facility's longest horizontal dimension is less than 50 ft. If the facility's longest dimension is longer than 50 ft, then one test should be conducted for every 50 ft of facility (rounded up) or every 2,000 ft^2 of facility (rounded up), whichever is highest. When possible, multiple tests may be conducted simultaneously to minimize the cost of testing. The equation for determining the number of tests is:

$$T = \text{Roundup}\left(\text{Maximum}\left[\frac{D_{max}}{50}; \frac{AREA_{\max}}{2{,}000}\right]\right) \quad (1)$$

Where: T = Number of tests; D_{max} = longest horizontal ponding dimension of infiltration facility (ft). To the extent possible, the test locations should be evenly spaced within the footprint of the proposed facility.

If the location and elevation of the proposed infiltration facility is not known in advance, or it is infeasible to conduct testing within the footprint and at the same elevation as the proposed infiltration facility, testing may be conducted in the vicinity of the proposed facility or at a slightly different elevation. In these scenarios, the K_b estimate provided by the test should be multiplied by an appropriate uncertainty correction factor (CF_u), as discussed in Section 5.2.

Horizontal infiltration facilities, such as bioretention facilities, infiltration ponds and permeable pavement, are dominated by vertical flow out of the bottom of the facility. In order to replicate the vertical flow, it is preferred to conduct infiltration tests in dug pits rather than boreholes or wells. However, properly designed borehole tests may be used to estimate K_b for horizontal infiltration facilities when dug pits are infeasible or overly disruptive to existing infrastructure. In particular, borehole tests may be preferred at sites with significant grading when the bottom elevation of the proposed infiltration facility is significantly deeper than the existing ground surface. In addition, borehole tests may be easier in areas with pavement, utilities, and tight spaces that are difficult to access with an excavator. As discussed later in the Guide, K_b estimates provided by borehole tests should be multiplied by the test well correction factor (CF_w) when used for sizing horizontal infiltration facilities.

3.2 Test Design for Drywells

The SWMMWW provides design details for two types of drywells. The drywell described in Section V-4 (BMT T5.10A) is a gravel-filled excavation at least 4 ft deep and 4 ft wide with no perforated pipe or manhole structure. The drywell described in Section V-5 (BMT T7.50) is a gravel-filled excavation at least 9 ft deep and includes a perforated concrete manhole structure. Although not included in the SWMMWW, drilled drywells are common in the southwestern United States and are becoming more common in Washington State. For purposes of this Guide, drywells are any excavated or drilled infiltration facility with $r_e < H_{max}$. The capacity of drywells is dominated by horizontal flow out the sides of the facility. Ideally, the location and vertical infiltration interval of the proposed drywell are known in advance of infiltration testing.

For a dug drywell, the infiltration interval extends from the bottom of the excavation to the maximum ponding elevation, which is usually determined by the outflow invert. If there is no outflow or surface seal, the maximum ponding elevation for a dug drywell corresponds to the top of the gravel in the drywell. For a drilled drywell with a surface seal, the infiltration interval corresponds to the filter pack interval of the drywell.

In order to replicate the flow dynamics of the proposed drywell, infiltration testing should be conducted as close as possible to the proposed drywell location and the test facility should be designed to deliver water to native soils across the proposed infiltration interval. However, the exact location and infiltration interval of the proposed drywell may not be known in advance. In many cases, the infiltration interval will be based on subsurface conditions observed during drilling. For example, the bottom of the glacial till may determine the top of the infiltration interval and the water table may determine the bottom of the infiltration interval.

Test design for drywells should consider the separation from groundwater and low permeability perching layers outlined in Section 2.3. As discussed in Section 3.3.3, in some cases it may be necessary to install groundwater monitoring wells to determine the seasonal high groundwater table.

3.3 Field Procedures

Field procedures for three test methods are provided in this section, including: 1) excavated pit test, 2) uncased shallow borehole test, and 3) deep test well. All of these borehole permeameter (BP) test methods rely on the methods outlined by Kindred (2022, Volumes I and II) and demonstrated in the field by Kindred (2022, Volumes IV and V).

3.3.1 Field Procedures – Excavated Pit Test

K_b estimates from steady-state infiltration testing in excavated pits are the preferred method for sizing horizontal infiltration facilities. A photograph of an infiltration test in an excavated pit is provided in Fig. 1. Pit tests are conducted in the following manner:

1) The test pit should be located in the same soils as the proposed infiltration facility. In previously developed sites, care should be taken to locate the test pit far enough from utilities so the utility trench backfill does not interfere with test results.

2) The pit may be either hand-dug or machine-excavated. The bottom area of the pit should be between 12 and 100 ft^2 and the side walls should be sloped or benched to maintain side-wall stability. The bottom of the pit should be as flat as possible and generally vary less than ±0.2 ft over the majority of the excavation. Small accumulations of sluff around the edges of the pit are acceptable and will not significantly affect the results.

3) Document the soil and groundwater conditions observed in the pit. Particular attention should be paid to document the transition from loose, weathered, surface soils to denser, unweathered, deeper soils. Soil samples may be collected from the bottom of the pit and delivered to a soil testing laboratory for moisture content and grainsize analyses. These results may be useful for documenting that the tested soils are similar in texture to the soils in the base of the proposed infiltration facility.

4) Measure the length and width of the excavation at the bottom and 1 foot above the bottom of the pit (the approximate maximum depth of ponding). Record these values.

5) Record the depth of the pit (may vary depending on ground slope). The bottom elevation of the test pit should be as close as possible to the bottom elevation of the proposed facility, ideally within 1 ft. However, the elevation of the test excavation/boring should be selected such that the ponding interval during the test does not intersect with a more permeable soil layer above the bottom of the proposed infiltration facility. This is a common issue for shallow infiltration facilities because near-surface soils are typically loosened due to bioturbation and weathering and these surface soils will likely be compacted or removed during

construction. It may be necessary to deepen the test excavation/boring below the bottom of the proposed facility to minimize water flowing out into the loose surficial soil horizon. The goal is to obtain an accurate estimate of K_b for the soils exposed in the base of the proposed infiltration facility.

6) If available, place a pressure transducer in the bottom of the excavation in a location that represents the average depth of the excavation. The pressure transducer should be set to record the water depth once per minute. The pressure transducer may be connected to a data cable that allows real-time monitoring of the depth of water during the test. If the transducer is not vented, the transducer data should be corrected for atmospheric pressure using data from a barometric transducer. The analysis of the results will be based on the transducer data, when available.

7) Place a vertical stadia rod marked in increments of 1/10th of a foot (or smaller). The bottom of the stadia rod should be placed in a location that represents the average depth of the excavation. Record the ponding depth on the stadia rod at least once every 15 min. during the steady-state portion of the test. The analysis of the results may be based on the stadia rod measurements if transducer data is not available.

8) Water may be provided using a hose bib, a water truck, or a fire hydrant. Hose bibs are generally limited to 5-10 gpm, water trucks are generally limited to 80 gpm, and fire hydrants are generally limited to 140 gpm. Most pit tests can be completed with a flow rate of 20 gpm or less (7,200 gallons for a 6-hr test).

9) Discharge water into the excavation through a short section (<5 ft) of slotted polyvinyl chloride (PVC) screen or a permeable sack in the bottom of the pit to reduce erosion and disturbance of the bottom soils. Fill the excavation to a target depth of between 0.5 and 1.0 ft. Adjust the flow rate to maintain a constant depth of water (±0.1 ft) for a minimum of 6 hours.

10) When feasible, use a flow meter to measure the rate of water flow into the excavation. At low flow rates (generally less than 0.75 gpm) the flow may be less than the calibrated range of the flow meter. In this situation, the flow rate may be estimated based on the amount of time it takes to fill a container of known volume. Record the flow rate at least once every 15 min.

11) At very low flow rates (generally less than 0.1 gpm) the metering valve may no longer be capable of adjusting the flow rate. In this situation, water should be added every time the water level falls 0.1 ft below the steady-state target depth. The amount of water added should be measured using a flow meter. The flow rate is estimated by dividing the amount of water by the number of minutes between refills.

12) At least once during the test, the accuracy of the flow meter shall be checked by measuring the time to fill a container of known volume. The manual flow rate shall be within 10% of the meter flow rate. If not, the meter flow rates should be adjusted to account for the inaccuracy.

13) After maintaining a constant head elevation for 6 hours, turn the water off and record the ponding depth during the falling-head portion of the test. The frequency of measurement depends on the rate of falling head. Ideally, the change in ponding depth between measurements is less than 0.05 ft. If the rate of falling head is very slow, the transducer should be left in the excavation to record the ponding depth until the excavation is dry.

14) Once the water is fully drained from the pit, excavate an exploration pit or borehole at least 3 ft deeper than the pit bottom to observe and document soil and groundwater conditions. The excavation may be conducted either inside or outside the test pit. A combination of hand tools, including hand-auger, posthole digger, and shovel, may be necessary to excavate to a depth of 3 ft below the bottom of the test pit. In accordance with OSHA regulations, do not climb into the pit if it is deeper than 4-ft or if there is evidence of sidewall caving. In some cases, an exploration pit or borehole 3 ft below the bottom of the test pit is not feasible due to cobbles and/or caving.

15) Once testing and soil sampling is complete, backfill the test pit with material and compact to the desired level of compaction.

16) Evaluate the results using the uncased steady-state borehole permeameter (USSBP) method provided in Section 4.

Fig. 1: Photograph of an infiltration test in an excavated pit.

3.3.2 *Field Procedures – Uncased Shallow Borehole Test*

Shallow, steady-state borehole tests may be conducted in temporary test wells that are less than 10 ft deep. The K_b estimates provided by these tests may be used to size horizontal infiltration facilities or drywells that are less than 10 ft deep. Since shallow infiltration facilities are generally designed to operate as uncased facilities (i.e., the water level doesn't rise up into a solid casing), shallow borehole tests are generally designed to be uncased, as described below:

1) The borehole should be located in the same soils as the proposed infiltration facility. In previously developed sites, care should be taken to locate the borehole far enough from utilities so the utility trench backfill does not interfere with test results.

2) Excavate the borehole to the desired depth using either a hand-auger, a vactor truck, a drill rig, or an excavator. Vactor-excavated boreholes have been excavated in glacial till and cobbly soils and generally provide a clean borehole ideally suited for testing. Hand-augered and machine-drilled boreholes can be difficult to complete in gravelly soils and sidewall smearing in silty soils may restrict water flow into more permeable layers.

3) Excavated test facilities are essentially deep pits with washed rock to maintain sidewall stability. Area of the pit should be as small as possible given the depth of the hole and the limitations of the equipment.

4) If feasible, extend the borehole at least 5 ft deeper than the bottom elevation of the proposed infiltration facility. This will provide soil and groundwater information below the proposed facility. Collect soil samples at least every 1.0 ft during drilling. Document the soil and groundwater conditions observed in the borehole. Record the depth and diameter of the borehole.

5) It may be necessary to backfill the borehole with bentonite chips (or comparable sealing material) to achieve the desired test interval. In order to prevent coating the borehole sidewalls with bentonite dust, the bentonite chips should be placed through a drop pipe to the desired elevation.

6) Obtaining a representative estimate of K_b using borehole tests requires careful selection of the test interval, as illustrated in Fig. 2. Borehole tests shall be conducted with 3-4 ft of ponding head. The goal is to obtain an accurate estimate of K_b for the soils exposed in the base of the proposed infiltration facility. Therefore, the bottom of the test interval should be as close as possible to the bottom of the proposed infiltration facility, subject to the following constraints:

   a. As shown in Fig. 2(a), the test interval must be below the loose weathered zone near the ground surface. This is to avoid artificially elevating the K_b estimate due to flow into these surface soils that will likely be compacted or removed during construction. Therefore, it may be necessary to deepen the test boring below the bottom of the proposed facility to minimize water flowing out into the surface soil horizon and maintain 3-4 ft of ponding during the test.

   b. The test interval must be below the maximum ponding elevation of the proposed facility. As shown in Fig. 2(b), if the proposed facility is designed to have less than 3 ft of ponding depth, then the top of the test interval shall align with the maximum ponding elevation in the proposed facility.

   As shown in Fig. 2(c), if the proposed facility is designed to have more than 3 ft of ponding depth, then the bottom of the test interval shall align with the bottom of the proposed facility.

7) As shown in Fig. 3, the temporary test well should be constructed in the shallow borehole using 2-in. diameter PVC well screen and casing. Place well screen of the desired length attached to blank PVC casing in the borehole. The top of the blank PVC casing should extend higher than the ground surface. The bottom of the screen section should be capped but a small hole (~1/8-in. diameter) should be drilled in the cap to let the water drain out. Record the length and diameter of screen and solid casing and the height of the solid casing above the ground surface.

8) Create a filter pack by placing coarse sand or pea gravel in the annular space around the PVC screen to maintain sidewall stability. The filter pack should extend above the desired test interval. Record the volume of sand or gravel added to the borehole.

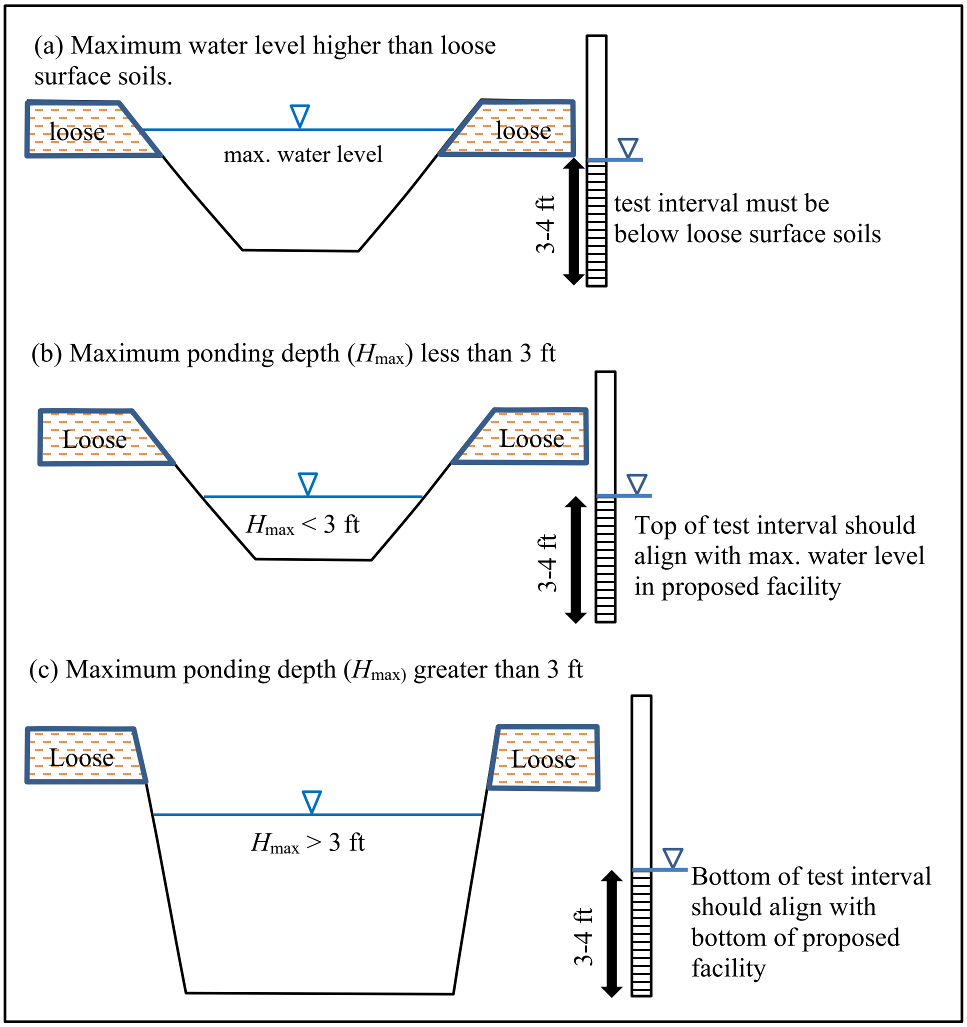
9) Place a pressure transducer in the bottom of the PVC screen. The pressure transducer should be set to record the water depth once per minute. The pressure transducer shall be connected to a data cable that allows real-time monitoring of the depth of water during the test. If the transducer is not vented, the transducer data should be corrected for atmospheric pressure using data from a barometric transducer. The analysis of the results will be based on the transducer data.

10) Water may be provided using a hose bib, a water truck, or a fire hydrant. Hose bibs are generally limited to 5-10 gpm, water trucks are generally limited to 80 gpm, and fire hydrants are generally limited to 140 gpm. Most tests can be completed with a flow rate of 20 gpm or less (7,200 gallons for a 6-hr test duration).

11) If the flow rate is greater than 5 gpm, water should be discharged into the test well using a drop pipe to minimize turbulent flow. The diameter of the drop pipe should be small enough to allow the electronic water level probe to pass between the drop pipe and the inside of the well casing. Alternatively, a second

pipe may be inserted into the borehole (before addition of the filter pack) for the electronic water level probe.

Fig. 2: Examples of borehole test intervals relative to planned infiltration facility.



12) Measure the depth to water from the top of casing using an electronic water-level tape approximately once per minute until the water level and flow rate have generally stabilized (usually about 30 minutes). Continue to manually measure the water level every 15 min. during the remainder of the test.

13) The flow rate shall be adjusted to maintain a steady-state water level with a ponding depth between 3-4 ft.

14) Record the flow data at least once every 15 min. When feasible, use a flow meter to measure the rate of water flow into the test well. At low flow rates (generally less than 0.75 gpm) the flow may be less than the calibrated range of the flow meter. In this situation, the flow rate may be estimated based on the amount of time it takes to fill a container of known volume.

15) At very low flow rates (generally less than 0.1 gpm) the metering valve may no longer be capable of adjusting the flow rate. In this situation, water should be added every time the water level falls 0.5 ft below

the steady-state target depth. The amount of water added should be measured using a flow meter. The flow rate is estimated by dividing the amount of water by the number of minutes between refills.

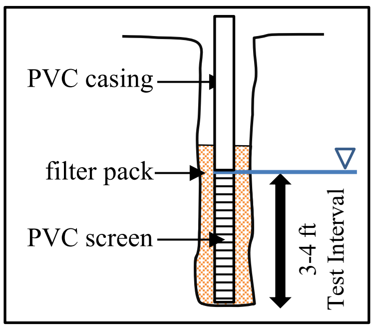
16) At least once during the test, the accuracy of the flow meter shall be checked by measuring the time to fill a container of known volume. The manual flow rate shall be within 10% of the meter flow rate. If not, the meter flow rates should be adjusted to account for the inaccuracy.

17) To the extent possible, maintain a constant head elevation in the test well for 6 hours. After 6 hours, turn the water off and record the ponding depth during the falling-head portion of the test using the transducer. The transducer should be left in the test well to record the ponding depth until the test well is dry.

18) When testing is complete, remove the PVC screen and casing and backfill the hole with gravel and/or soil.

19) Evaluate the results using the USSBP method provided in Section 4.

Fig. 3: Schematic example of temporary test well.



3.3.3 *Field Procedures – Deep Test Wells*

Test wells deeper than 10 ft must be constructed by a licensed driller and in accordance with Washington Administrative Code (WAC) 173-160 (Minimum Standards for Construction and Maintenance of Wells). The K_b estimates provided by steady-state infiltration testing in these wells are well suited for estimating the capacity of drywells deeper than 10 ft. They may also be used for sizing horizontal infiltration facilities that will be constructed more than 10 ft below the current ground surface. Infiltration tests in deep test wells are conducted in the following manner:

1) Drill the borehole to the target depth using a drilling method that can provide a clean hole. Sonic drilling and air-rotary generally provide good test wells, although there may be borehole smearing issues if wet silty soils are penetrated during drilling. If borehole smearing is a concern, it may be necessary to place a larger casing through the silty interval and drill deeper using smaller diameter casing. When drilling with sonic methods, a slower drilling rate is preferable so as not to bake dense or gravelly soil and potentially grind the soil into dust, which could be smeared on the borehole wall.

2) Wells drilled using hollow-stem-auger methods have exhibited clogging or side-wall smearing, although this drilling method may be suitable in very clean soils. Mud rotary is not recommended due to the difficulty of removing the drilling mud from wells completed above the water table.

3) Soil and groundwater information should be collected to a depth of at least 10 ft below the bottom of the proposed drywell if the drywell does not penetrate an upper confining layer and to a depth of at least 20 ft below the proposed drywell if it does penetrate an upper confining layer. This will allow collection of soil and groundwater data required to assess groundwater separation requirements discussed in Section 2.3 and support mounding analysis, if necessary. The borehole should be backfilled with bentonite pellets up to the bottom of the desired test interval. A minimum of 6 in. of filter material should be installed above the

bentonite pellets before placing the well screen (i.e. the test well screen should not be installed directly on the bentonite pellets).

4) If groundwater water is observed near the base of the boring, a pressure transducer or vibrating wire piezometer can be installed below the water table to monitor potential groundwater mounding during testing.

5) Collect soil samples during drilling and document the soil and groundwater conditions observed in the borehole. Record the depth and diameter of the borehole.

6) Obtaining a representative estimate of K_b using borehole tests requires careful selection of the test interval. If the test will be used to design a horizontal infiltration facility, the test should be conducted as an uncased test and the test interval should be 3-4 ft long using the same approach provided for the uncased shallow borehole test. If the test will be used to predict the performance of a drywell, the test interval should be designed to match the filter pack interval for the proposed full-scale facility.

7) The well should be constructed using either 4-in. or 2-in.-diameter PVC well screen and casing. The 4-in.-diameter material allow use of a 2-in.-diameter drop pipe (for higher flow rates) and use of an electronic water-level tape. Place well screen of the desired length attached to blank PVC casing in the borehole. The bottom of the screen section should be capped but a small hole (~1/8-in diameter) should be drilled in the cap to let the water drain out. Record the length and diameter of screen and solid casing and the height of the solid casing relative to the ground surface.

8) Create a filter pack by placing coarse sand or pea gravel in the annular space around the PVC screen and at least 0.5 ft above the top of the screen. The filter pack should extend to the top of the desired test interval. Record the volume of sand or gravel added to the borehole.

9) Well development should be conducted to remove sediment and settle the filter pack. It is difficult to develop wells completed above the water table as it is necessary to add and remove water simultaneously or by alternating between adding water and removing water. Experience indicates that a small submersible pump is best for removing water while adding water.

10) Place a pressure transducer in the bottom of the PVC screen. In order to avoid damaging the pressure transducer, it should have a range that is greater than the depth of the well or the maximum water depth during the test. The pressure transducer should be set to record the water depth once per minute. The pressure transducer should be connected to a data cable that allows real-time monitoring of the depth of water during the test. The analysis of the results will be based on the transducer data.

11) If possible, measure the depth to water from the top of casing using an electronic water-level tape approximately once per minute until the water level and flow rate have generally stabilized (approximately 30 minutes). Continue to manually measure the water level every 15 min. during the remainder of the test.

12) Flow rates for deep infiltration tests are typically quite high and the tests are generally conducted using a fire hydrant. In some cases, the maximum flow from a fire hydrant (generally about 140 gpm) is not sufficient to fully saturate the desired test interval. If a fire hydrant is not available, it is possible to conduct the test with multiple water trucks with a maximum flow rate of 80 gpm. The test may underestimate K_b of the proposed drywell if is not possible to fully saturate the test interval during the test. An over-conservative design at high flow rates is considered acceptable.

13) If the flow rate is greater than 5 gpm, water should be discharged into the test well using a drop pipe to reduce turbulent flow. A 1-in. drop pipe can be used in 2-in.-diameter well casing, although the small diameter pipe will restrict the flow at higher flow rates. Higher flows can be achieved using a 2-in. drop pipe. The drop pipe should extend below the water level during the test to reduce air-entrainment.

14) The flow rate shall be adjusted to maintain the water level at the desired depth. Ideally, the water level shall be near the maximum ponding depth in the planned drywell. However, in permeable formations and deep

wells, it may not be possible to achieve this water level given limitation on the water supply. This will result in a conservative estimate of K_b.

15) Record the flow data at least once every 15 min. When feasible, use a flow meter to measure the rate of water flow into the test well. At low flow rates (generally < 0.75 gpm) the flow may be less than the calibrated range of the flow meter. In this situation, the flow rate may be estimated based on the amount of time it takes to fill a container of known volume.

16) If possible, the accuracy of the flow meter shall be checked by measuring the time to fill a container of known volume. This will require disconnecting the drop pipe while filling the container and can be difficult at high flow rates. If manual measurement of the flow rate is feasible, the manual flow rate shall be within 10% of the meter flow rate. If the manual flow rate is not within 10% of the meter flow rate, the meter flow rates should be adjusted to account for the inaccuracy. If it is not feasible to manually check the flow rate, a flow correction factor of 0.9 should be applied (see Section 5.1).

17) After maintaining a constant head elevation for 6 hours, turn the water off and record the ponding depth during the falling-head portion of the test using the transducer. The transducer should be left in the test well to record the ponding depth until the test well is dry.

18) The test well should be properly decommissioned in accordance with WAC 173-160 when it is no longer needed.

19) Evaluate the results using the USSBP or CSSBP methods provided in Section 4.

4 Calculating Bulk Hydraulic Conductivity (K_b)

Methods for evaluating field testing results and demonstrations of these methods are provided by Kindred (2022). K_b can be estimated from the field-testing results using either the USSBP equation for uncased tests, or the CSSBP equation for cased tests. As discussed in Kindred (2022, Volume I) and Kindred and Reynolds (2020), the USSBP equation is used for tests in excavated test pits and test wells where $H < 1.2$, as follows:

$$K_\mathrm{b} = \frac{C_\mathrm{u} Q}{2\pi H^2 + \pi r_\mathrm{e}^2 C_\mathrm{u} + \frac{2\pi \mathrm{H}}{\alpha^*}} \quad (2)$$

where: r_e (ft) is equal to the borehole radius, or, for tests in excavated test pits:

$$r_e = \sqrt{pit\ width \times pit\ length/\pi} \quad (3)$$

When $H > 1.2\ L$, K_b is estimated using the CSSBP equation (Kindred 2022, Volume II):

$$K_\mathrm{b} = \frac{C_\mathrm{c} Q}{2\pi LH + \pi r_\mathrm{e}^2 C_\mathrm{c} + \frac{2\pi L}{\alpha^*}} \quad (4)$$

where:

H (ft) = ponding head at the end of the steady state test

Q (ft^3/d) = flow rate at the end of the steady state test

C_u = uncased shape factor

α^* (ft^{-1}) = soil sorptive number

L (ft) = length of the filter pack

C_c = cased shape factor

H, L, Q, and r_e are all based on the dimensions of the test facility and the results of the infiltration test. The procedures to estimate α^*, C_u and C_c are provided in the sections below.

4.1 Estimating the Soil Sorption Number (α^*)

α^* represents the capillarity or suction capacity of the soil and can be visualized as the inverse of the wetting front height above the water table. Capillarity is a function of the size of pore spaces in the soil, whereby smaller pore spaces have greater capillarity. In general, fine-grained soils have stronger capillarity (lower α^*) and coarse-grained soils have weaker capillarity (higher α^*). In addition, well-graded (poorly sorted) soils (soil particles with a wide range of grainsizes) have stronger capillarity than poorly-graded (well sorted) soils (soil particles with a uniform grainsize). This is because the smaller soil particles fill the spaces between the larger soil particles, resulting in smaller pores.

α^* has been estimated for ten soils types representing the types of soils typically considered for infiltration in Washington State (Kindred 2022, Volume I). These ten soil types include five glacially consolidated soils (glacial till and four advance outwash soils) and five normally consolidated soils ranging from silty fine sand to sandy gravel. Table 1 (normally consolidated soils) and Table 2 (glacially consolidated soils) provides the soil characteristics and associated α^* for each of the ten representative soils. Selection of α^* should be based on the soil characteristics and USCS class that best represents the soils in the tested interval. Generally, grainsize analyses of the soils in the tested interval, combined with an estimate of the degree of compaction, provides the necessary soil characteristics to select α^*.

In scenarios where the tested interval includes multiple representative soil types, α^* should be selected based on the dominant soil type. If there is not a dominant soil type, or the dominant soil falls between two or more representative soil types, the finer-grained soil type (i.e., the one with the lowest α^*) should be selected. Examination of Eq. (1) shows that assuming a lower α^* will provide a more conservative (lower) estimate of K_b.

Table 1 Properties of representative glacially-consolidated soil types used to estimate α^*.

Parameter	Soil Type				
	Qvt	**Silty Qva**	**Fine Qva**	**Fine-Medium Qva**	**Fine-Coarse Qva**
D_{60} (mm)	0.5	0.25	0.3	0.5	5
D_{10} (mm)	0.02	0.04	0.1	0.13	0.25
Silt Content (wt. %)	20	17	8	5	3
USCS Soil Type	SM	SM	SM-SP	SP	SW
Sorptive Number α^* (ft^{-1})	0.36	0.41	0.76	1.2	7.6

Notes: *Qva* - advance outwash
Qvt - glacial till
D_{60} and D_{10} are grain diameters corresponding, respectively, to 60% passing and 10% passing on the grain-size distribution curve
USCS - Unified Soil Classification System.

Table 2 Properties of representative normally-consolidated soil types used to estimate α^*.

Parameter	Soil Type				
	Silty Fine Sand	**Silty Fine-Coarse Sand**	**Fine Sand**	**Medium Sand**	**Sandy Gravel**
D_{60} (mm)	0.15	1.4	0.28	1.0	8.0
D_{10} (mm)	0.04	0.02	0.079	0.18	0.4
Silt Content (wt. %)	25%	15%	9%	5%	3%
USCS Soil Type	SM	SM	SM-SP	SP	GW
Sorptive Number α^* (ft^{-1})	0.5	1.7	1.1	3.4	17.4

Notes: D_{60} and D_{10} are grain diameters corresponding, respectively, to 60% passing and 10% passing on the grain-size distribution curve.
USCS - Unified Soil Classification System.

4.2 Estimating the Shape Factor (C_u or C_c)

The uncased shape factor (C_u) and the cased shape factor (C_c) are dimensionless, empirical fitting parameters that vary depending on the geometry of the infiltration test, quantified as H/r_e or L/r_e. C_u and C_c increase as the H or L increases relative to r_e and are calculated using:

$$C_{u/c} = \left[\frac{\left(H/r_e\right)}{Z_1+Z_2\left(H/r_e\right)}\right]^{Z_3} \tag{5}$$

Values for Z_1, Z_2, Z_3 are different for C_u and C_c and are provided in Table 3 and Table 4, respectively. They were estimated based on a calibration process (Kindred 2022, Volumes I and II). Four sets of Z parameters are provided for uncased and cased scenarios, based on silt content and the ratios H/r_e and L/r_e.

The USSBP method is calibrated for uncased scenarios where $H = L$ and the CSSBP method is calibrated for cased scenarios when $H > L$. However, there is a transition interval as the water level begins to rise above the filter pack into the solid casing above the filter pack. Based on simulations conducted by Kindred (2022, Volume II) the USSBP method is recommended when $H/L < 1.2$ and the CSSBP method is recommended when $H/L > 1.2$

Table 3 Uncased s*hape function* (C_u) parameters for USSBP tests based on different soil classifications using the Unified Soil Classification System.

Soil Type	**Low Ponded Head ($H/r_e \leq 20$)**			**High Ponded Head ($H/r_e \geq 20$)**		
	Z_1 **(-)**	Z_2 **(-)**	Z_3 **(-)**	Z_1 **(-)**	Z_2 **(-)**	Z_3 **(-)**
Sand and gravel with > 12% Silt (SM, GM)	2.11	0.192	0.91	2.04	0.0224	0.547
Sand and gravel with < 12% Silt (SP-SM, SP, SW, GW, GP)	2.03	0.207	0.98	2.11	0.0273	0.605

Table 4 Cased s*hape function* (C_c) parameters for CSSBP tests based on different soil classifications using the Unified Soil Classification System.

Soil Type	**Short Filter pack ($L/r_e < 20$)**			**Long Filter pack ($L/r_e \geq 20$)**		
	Z_1 **(-)**	Z_2 **(-)**	Z_3 **(-)**	Z_1 **(-)**	Z_2 **(-)**	Z_3 **(-)**
Sand and gravel with > 12% Silt (SM, GM)	3.06	0.12	0.674	2.32	0.0286	0.463
Sand and gravel with < 12% Silt (SP-SM, SP, SW, GW, GP)	2.45	0.214	0.93	1.87	0.0354	0.501

4.3 Multiple Test Results for a Single Infiltration Facility

In some cases, there may be multiple infiltration tests within or near the footprint of the proposed facility. When the soil within the footprint appears to be relatively uniform, it may be suitable to simply average the test results to generate a K_b for use in design. If site characterization is able to delineate the distribution of multiple soil types within the facility, the K_b used for design should be calculated based on the K_b estimates for each soil type and the percentage of each soil type within the facility footprint.

5 Calculating Design Hydraulic Conductivity (K_d)

For a variety of reasons, a full-scale infiltration facility may perform differently than predicted based on K_b provided by an infiltration test. Potential reasons include the following:

1) Analytical error in the BP methods used to estimate K_b (Kindred 2022, Volumes I and II).
2) Uncertainty in Q due to inability to conduct manual verification of flow rate.
3) The proposed infiltration facility may be in a different horizontal and/or vertical position that the infiltration test facility. This can change the effective K_b due to horizontal and vertical variability in soil characteristics (Kindred 2022, Volumes IV and VI).
4) Full-scale infiltration facilities are generally larger than the test facilities and may infiltrate more water than used during infiltration testing, which may increase the potential for groundwater mounding (Kindred 2022, Volume VI).
5) The proposed infiltration facility may be a different shape than the test facility and the flow dynamics (vertical dominated versus horizontal dominated) may be different. Because of layering and groundwater mounding, this may change the effective K_b (Kindred 2022, Volume VI).
6) The infiltration capacity of the facility may change over time due to clogging.

In order to address these potential differences, K_b is multiplied by a number of correction factors to arrive at the design hydraulic conductivity (K_d). The following equation is used to calculate K_d:

$$K_d = K_b \times CF_f \times CF_r \times CF_u \times CF_w \times CF_m \times CF_c \tag{6}$$

Where:

CF_f = flow correction factor

CF_r = recharge correction factor

CF_u = uncertainty correction factor

CF_w = well correction factor

CF_m = mounding correction factor

CF_c = clogging correction factor

The technical basis and recommended values for each of these correction factors are discussed below.

5.1 Flow Correction Factor (CF_f)

As discussed in Section 3.3.3, it may be difficult to conduct a manual verification of the flow rate at very high flow rates. At flow rates over 50 gpm, readily-available containers are filled too quickly to obtain an accurate estimates of flow. For example, at 100 gpm, a 5-gallon bucket is filled up in 3 seconds. Since the goal is to confirm that the meter flow rate has less than 10% error, the manual flow rate should be accurate within ±5%. As a general rule of thumb, the time to fill the container should be at least 20 seconds to obtain a reasonably accurate flow measurement.

If a reasonably accurate manual verification of the meter flow rate is available, and the meter flow rate has been adjusted accordingly, then CF_f = 1.0. If a reasonably accurate manual verification of the meter flow rate is not available, then CF_f = 0.9.

5.2 Recharge Correction Factor (CF_r)

Kindred (2022, Volume I) demonstrated that the background moisture content of the soils could influence the capillary flux and the estimated K_b for fine-grained soils. Since an operational infiltration facility will deliver significantly more groundwater recharge than background recharge conditions, it is likely that the background moisture content below an operational infiltration facility will be higher during the wet season than under typical test conditions. Higher background moisture means less capillary suction, resulting in less flow (and a lower K_b) than measured during an infiltration test.

Numerical simulations (Kindred 2022, Volume I) demonstrated that infiltration tests in soils with more than 12% silt could underestimate K_b for an operational infiltration facility by 5% to 19% for H/r_e = 1.0 and 5% to 9% for a H/r_e = 100. This difference will be addressed using the recharge correction factor (CF_r). Table 5 provides recommended values for CF_r for different soil types and H/r_e ratios.

Table 5 *Recharge correction factor* (CF_r) for different soil types and H/r_e ratios.

Soil Type	H/r_e Ratio		
	<0.3	0.3-3	>3
Qvt	0.7	0.8	0.95
Silty Qva	0.95	0.95	0.95
F Qva, F-M Qva, F-C Qva	1.0	1.0	1.0
Silty F Sand	0.7	0.8	0.9
Silty F-C Sand	0.9	0.9	0.9
F Sand, M Sand, sandy gravel	1.0	1.0	1.0

Notes: Qvt = glacial till, Qva = advance outwash, F =fine, M = medium, C = coarse

5.3 Uncertainty Correction Factor (CF_u)

The uncertainty correction factor (CF_u) is intended to address both analytical error and field variability. Based on over 400 calibration simulation conducted by Kindred (2022 Volume I and II) the analytical error of the BP methods used in this guide had a maximum error of 13% and an average error of 4%. An average error of ±4% suggests a CF_u of 0.96 would address analytical error. Comparison of field infiltration testing results suggests a median CF_u of 0.56.and a 20th percentile of 0.19 (Kindred 2022, Volume IV), which demonstrates that the field variability is much greater than the analytical error.

The uncertainty in K_b due to layering and spatial variability means that test results may either under-predict or over-predict the performance of full-scale facilities. If the project includes numerous small facilities, it is likely that the over-estimates of performance will tend to balance the under-estimates of performance and the overall project is likely to achieve the overall flow-control objectives. small site with relatively minor consequences However, if the consequences of under-predicting facility performance are significant (e.g., will result in flooding or erosion) then it may be appropriate to apply an uncertainty correction factor (CF_u) that will reduce the likelihood of under-predicting facility performance.

If the test is conducted more than 20 ft from the proposed facility and the tested infiltration interval is more than 10% higher or lower than the proposed infiltration interval, the K_b estimate provided by the test should be multiplied by an appropriate uncertainty correction factor (CF_u). A CF_u in the range of 0.2 to 0.5 may be appropriate for higher-risk infiltration facilities with limited infiltration test data. A higher CF_u (not to exceed 1) may be appropriate if numerous infiltration tests in the vicinity of the proposed infiltration facility are relatively consistent or if the consequences of under-predicting facility performance are insignificant (e.g., a small residential project with an offsite point of discharge).

5.4 Test Well Correction Factor (CF_w)

Due to stratigraphic layering (Kindred 2022, Volume VI), K_b estimates from test wells tend to overestimate K_b for horizontal infiltration facilities. The field testing conducted by Kindred (2012, Volume IV) provided a median CF_w of 0.53 and a 20th percentile of 0.32. Based on these results, a CF_w of 0.5 is recommended when well tests are used to size a horizontal facility (pond, bioretention facility, or permeable pavement). A CF_w of 1 is recommended if a pit test is used to size a horizontal facility or a well test is used to estimate the capacity of a drywell.

5.5 Groundwater Mounding Correction Factor (CF_m)

Groundwater mounding can reduce the capacity of an infiltration facility when the water table and/or a perching layer are a short distance beneath the facility. As demonstrated with numerical simulations (Kindred 2022, Volume VI) and observed in some infiltration tests (Kindred 2022, Volumes IV and V), 6-hour steady-state infiltration tests can detect the effects of groundwater mounding in some scenarios. Generally, if the test has reached steady-state, groundwater mounding during the test is relatively minor. Steady state is defined as a combined change in water level and flow rate that is less than 5% during the last hour of the test. If the combined change is greater than 5% during the last hour of the test, this is an indication that significant groundwater mounding is occurring.

If the proposed infiltration facility is relatively large compared with the test facility, or the project includes many small infiltrating facilities in close proximity, it is possible that site development may cause groundwater mounding that is not detected by an infiltration test (Kindred 2022, Volume VI). The potential for groundwater mounding should be evaluated and addressed appropriately for all projects with runoff from more than 10,000 ft^2 of impervious surface contributing to infiltration facilities. The requirements for this assessment are provided in Section 7.

For smaller sites (less than 10,000 ft^2 of impervious surface) a mounding assessment is generally not warranted. However, groundwater mounding may still impact the performance of an infiltration facility and it may be necessary to apply a mounding correction factor (CF_m). Kindred (2022, Volume VI) compared the mounding effects for horizontal infiltration facilities of different sizes with different soil types. For the example scenarios simulated in the study, CF_m varied from 0.2 to 1 depending on the soil K_b, the size of the facility, and the depth to groundwater. In general, CF_m decreases as the permeability of the soil increases, as the depth to groundwater decreases, and as the size of the facility increases.

Any estimate of CF_m needs to address evidence of groundwater mounding during the infiltration test as well as the soil type, the size of the facility and the depth to groundwater/perching layer. Therefore, the following equation is used to calculate CF_m for projects with less than 10,000 ft^2 of impervious surface draining to infiltration facilities:

$$CF_m = M_{test} \times M_{soil} \times M_{size} \times M_{depth} \tag{7}$$

The values for the mounding correction factors are provided in Table 5. If a mounding assessment is conducted and the potential for mounding is addressed explicitly based on a site-specific mounding assessment, then $CF_m = 1$.

Table 6 Groundwater mounding correction factors.

% Change in Flow Rate + water level in last hour of test	$\boldsymbol{M}_{\textbf{test}}$
<5%	1
5-10%	0.95
>10%	0.9

$\boldsymbol{K}_{\textbf{b}}$ **(ft/d)**	$\boldsymbol{M}_{\textbf{soil}}$
<2	1
2-10	0.95
>10	0.9

Impervious surface area contributing to infiltration	$\boldsymbol{M}_{\textbf{size}}$
0-2000	1
2,000 – 5,000	0.9
5,000 – 10,000	0.8
>10,000	Mounding Assessment

Depth to groundwater/perching layer (ft)	$\boldsymbol{M}_{\textbf{depth}}$
>10	1
5-10	0.9
<5	0.8

5.6 Clogging Correction Factor (CF_c)

Clogging of infiltration facilities is known to occur, although this phenomenon was not included in the scope of work for this study. A careful review of available research would be useful to better understand and quantify the potential for clogging of infiltration facilities. In the absence of this work, the recommendations for estimating the clogging correction factor (CF_c) provided in this Guide should be considered preliminary.

There are many examples of significant clogging associated with construction runoff, erosion, and high-traffic roads. For surface infiltration facilities, some or all of the infiltration capacity can be restored by removal of the accumulated sediment. However, for buried infiltration facilities, such as drywells and infiltration trenches, removal of the sediment (if feasible) may have limited benefit.

Another well-documented phenomenon is clogging of small-diameter (i.e., ≤ 24 in. diameter) drywells by biological growth when the runoff is treated with media containing nutrient-rich compost before discharge into the drywell. Nutrient-rich compost should not be used for treatment in advance of drywells.

In general, horizontal infiltration facilities with deep-rooting plants (e.g., bioretention facilities) appear to be less prone to clogging, likely due to root penetration and other bioturbation. There are reports that infiltration rates from bioretention facilities actually appear to increase over time. Therefore, bioretention facilities are considered less prone to clogging than unvegetated or grass-lined infiltration ponds.

Permeable pavements differ from other infiltration facilities because K_d doesn't generally affect the horizontal size of the facility, although it may affect the thickness of the detention layer when K_d is very low (generally less than 0.3 ft/d). Although the upper surface of permeable pavements is known to clog, clogging of the native soils below the permeable pavement facility has not been well studied. It is likely that the upper surface of the pavement retains much of the sediment and thus provides good pre-treatment. Although maintenance of permeable pavements is critical for preserving infiltration performance, it does not affect K_d of the native soils.

Large-diameter drywells (> 3 ft in diameter) have been in use for many decades and appear to be less prone to clogging than small-diameter drywells, likely due to their larger surface area and greater volume for sediment accumulation. However, these large drywells can be clogged with sediment if there is no pre-treatment and they are subject to heavy sediment loads. Pre-treatment (without a nutrient-rich media, as described above) is recommended to extend the life of drywells.

The factors that affect clogging include: 1) the traffic volume and likely sediment load, 2) the level of pre-treatment, 3) the ability to maintain the permeability of the infiltration facility, and 4) drywell diameter. Therefore, the following equation is used to calculate CF_c:

$$CF_c = CLOG_{load} \times CLOG_{pre} \times CLOG_{maint} \times CLOG_{dia} \quad (8)$$

The values for the clogging correction factors are provided in Table 6.

Table 7 Clogging correction factors.

Traffic/Sediment Load (Vehicles/d)	***CLOG***
Low (<100)	1
Moderate (100-1,000)	0.9
High (>1,000)	0.8

Level of Pre-Treatment	$CLOG_{pre}$
Bioretention Facility and Permeable	1
Filter Media (compost free)	1
Settlement sump/pond	0.9
None	0.8

Level of Maintenance (frequency of sediment removal)	$CLOG_{maint}$
Good (> once per year) and permeable	1
Moderate (every 1-3 years)	0.9
Poor (< every 3 years)	0.8

Drywell Diameter (in.)	$CLOG_{dia}$
>36	1
<36	0.8

5.7 Reporting

A summary of the infiltration assessment outlined in Section 2 through 5 shall be provided in an Infiltration Report. The Infiltration Report will include the following elements:

1. A summary of the proposed project and proposed infiltration facilities.
2. A description of the infiltration testing and subsurface investigations conducting at the site.
3. A description of surface and subsurface characteristics of the site.

4. An update of the feasibility assessment provided in the Preliminary Infiltration Feasibility Report (Section 2.5) based on any new subsurface information and updates of the proposed infiltration facilities.

5. Calculation of K_b (Section 4).

6. Recommendations for calculating K_d, including correction factors. Some of the correction factors, including CF_f, CF_r, M_{test}, and M_{soil}, are independent of the infiltration facility design and can be provided by the infiltration professional. Other correction factors, including CF_u, CF_w, CF_m, CF_c, M_{size}, M_{depth}, and all the clogging correction factors in Table 6, do depend on the configuration of the infiltration facility. The infiltration professional can provide values for these correction factors based on the preliminary infiltration facility design but they may be modified by the stormwater engineer if the design changes. The stormwater engineer should consult with the infiltration professional if the facility design is modified.

7. This report can be updated to include the groundwater mounding assessment (described in Section 7) if one is conducted.

6 Calculating Infiltration Capacity

In Washington State, stormwater management BMPs/facilities are usually designed and sized using continuous simulation runoff models that route runoff through the system using a time history of precipitation. The stormwater management BMPs/facilities are modified until the runoff from the site complies with regulatory requirements. In general, the goal is to reduce peak flows from the site to levels that do not cause environmental damage. The hydrologic models in common use include the Western Washington Hydrologic Model (WWHM) and MGS Flood.

The infiltration capacity of any infiltration facility can be estimated in these models using K_d and the dimensions of the infiltration facility by rearranging Eqs. 2 and 4 as follows:

when $H/L \leq 1.2$ use:

$$Q = \left(\frac{K_d}{C_u}\right)\left(2\pi H^2 + \pi r_e^2 C_u + \frac{2\pi H}{\alpha^*}\right) \tag{9}$$

or when $H/L > 1.2$ use:

$$Q = \left(\frac{K_d}{C_c}\right)\left(2\pi HL + \pi r_e^2 C_c + \frac{2\pi L}{\alpha^*}\right) \tag{10}$$

The infiltration capacity (Q) of horizontal infiltration facilities is calculated using Eq. 9 and the Q of drywells is calculated using either Eq. 9 or Eq. 10, depending on the ratio H/L. It's clear from these equations that Q is a function of ponding depth (H). Both of the hydrologic models listed above generally use an infiltration rate (I) and assume vertical flow out of infiltration facility to approximate Q. I has units of in./hr and is not dependent on H. Furthermore, the models assume no flow out of the sides of the infiltration facility. Simulating infiltration in this manner is appropriate for shallow infiltration facilities when the maximum H (H_{max}) in the facility is relatively small compared with the area of the facility. However, this approach is highly inaccurate for drywells and other facilities when H_{max} is large compared with the area of the facility.

Section 6.1 demonstrates how to estimate an infiltration rate based on K_d, an approach that is appropriate for horizontal infiltration facilities. Section 6.2 demonstrates how to incorporate H-dependent infiltration rates into the hydrologic models.

6.1 Horizontal Infiltration Facilities

For horizontal infiltration facilities, when $r_e > H_{max}$, hydrologic modeling can be conducted using an infiltration rate (I) and the ponded area of the facility. This is consistent with the traditional approach for simulating infiltration in Washington State. I is calculated using the following equation:

$$I = \frac{Q}{AREA_{\text{ave}}} \tag{11}$$

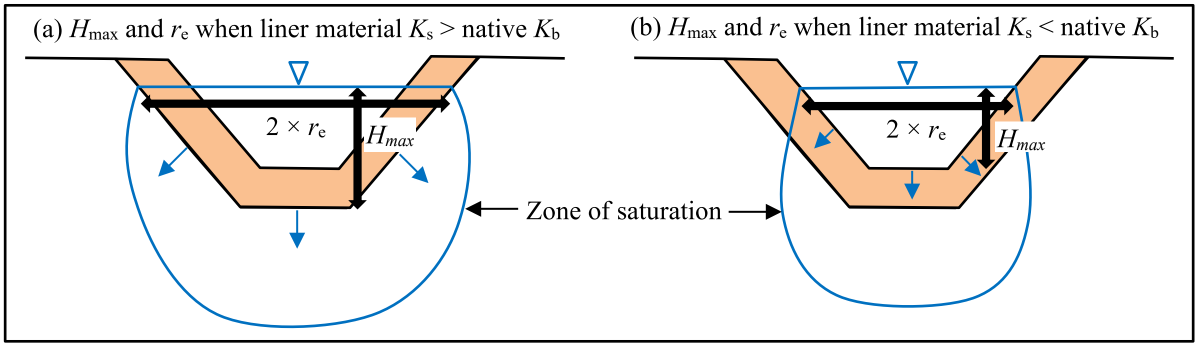
Where:

Q is calculated using Eq. 9 with $H = 0.5\ H_{max}$ and $r_e = \sqrt{AREA_{\text{ave}}/\pi}$

$AREA_{ave}$ = the ponded area of the facility when $H = 0.5H_{max}$.

If the infiltration facility is unlined or lined with materials that are more permeable than the native soil (Fig. 4a), $AREA_{ave}$, r_e, and H_{max} should be calculated assuming that the infiltration facility is defined by the interface with native soil. For example, if the facility includes 1.5 ft of treatment media and a ponding depth of 0.5 ft, H_{max} is 2.0 ft and $AREA_{ave}$ is calculated based on the dimensions of the excavation before treatment media is placed. If the infiltration facility is lined with materials that are less permeable than the native soil (Fig. 4b), $AREA_{ave}$, r_e, and H_{max} should be calculated assuming that Q and I are defined by ponding above the liner material and K_d of the liner material. This approach will provide a conservative estimate of Q and I since the native soils are more permeable than the liner material.

Fig. 4: H_{max} and r_e selection based on comparison of liner material K_s and native soil K_b.



6.2 Deep Infiltration Facilities

Deep infiltration facilities are defined as those where $r_e < H_{max}$, and could include deep infiltration ponds, bioretention facilities with a deep gravel-filled underdrain, and drywells. For these infiltration facilities, the hydrologic models should include a H-dependent infiltration rate.

For WWHM, H-dependent infiltration can be incorporated using the Stage-Storage-Discharge (SSD) Table element. This element should include the following columns: ponding depth, area, storage, and Q. The first three columns are calculated based on the dimensions and construction of the facility and Q can be calculated using either Eq. 9 or Eq. 10, depending on the H/L ratio.

For MGS Flood, this can be incorporated using a rating table, similar to the WWHM SSD Table element.

7 Groundwater Mounding Assessment

Groundwater mounding analyses may be necessary for two reasons: 1) to determine if groundwater mounding will limit the capacity of the infiltration facility, and 2) to assess the potential groundwater mounding impacts on steep slopes, structures, or surface flooding. In addition, local agencies may require mounding analysis is certain areas with known issues related to shallow groundwater. As described in Section 7.1, the groundwater mounding assessment may be limited to a screening-level assessment that relies on existing information and determines that groundwater mounding is not a concern. If the screening assessment determines that groundwater mounding may be a concern, additional data collection and groundwater modeling may be required, as described in Section 7.2.

7.1 Groundwater Mounding Screening Assessment

A screening-level groundwater mounding assessment is required for every project with more than 10,000 ft^2 of contributing impervious surface area. Generally, this assessment builds upon the desktop assessment and field feasibility activities outlined in Section 2.

7.1.1 Potential for Mounding to Limit Infiltration Capacity

The first purpose of the groundwater mounding screening assessment is to determine if groundwater mounding could potentially limit the capacity of infiltration facilities. This screening-level assessment is based on the distance between the bottom of the proposed infiltration facility and groundwater/perching layer and the amount of impervious surface draining to the infiltration facility or multiple infiltration facilities in close proximity. It is unlikely that groundwater mounding will limit the capacity of infiltration facility and more detailed assessment is not necessary if the following criteria are met:

1) < 1 acre of impervious surface draining to the infiltration facility (or multiple infiltration facilities within 100 ft of each other) and the groundwater/perching layer separation is greater than 10 ft.

2) 1-5 acres of impervious surface draining to the infiltration facility (or multiple infiltration facilities within 200 ft of each other) and the groundwater separation is greater than 20 ft.

3) > 5 acres of impervious surface draining to the infiltration facility (or multiple infiltration facilities within 400 ft of each other) and the groundwater separation is greater than 40 ft.

If these criteria are not met, a detailed groundwater mounding assessment is necessary to determine if groundwater mounding will limit the capacity of the infiltration facility. However, a groundwater mounding analysis may still be necessary to address the potential for adverse impacts, as discussed in the following section.

7.1.2 Potential for Adverse Impacts

The second purpose of the groundwater mounding screening assessment is to evaluate the potential groundwater mounding impacts on steep slopes, below-grade structures, and surface flooding. It is the responsibility of the infiltration professional to identify potential impacts that could result from groundwater mounding. For example, not all steep slopes are potential landslide hazards. In addition, potential impacts generally occur downgradient from the infiltration facility, although upgradient below-grade structures and surface flooding could be an issue if the ground surface is relatively flat.

These potential impacts are generally not dependent on the separation from groundwater/perching layer, but more dependent on groundwater mounding at the location of potential impact. Therefore, this screening assessment depends on two factors: 1) the distance between the infiltration facility and the location of potential impact and 2) the change in groundwater flux due to the project. Each of these are discussed below.

The appropriate distance between the infiltration facility and the location of potential impact that warrants detailed groundwater mounding assessment depends on the amount of stormwater infiltration, which is a function of the impervious area contributing to the infiltration facility. Table 7 specifies maximum distances for different values of impervious area. If the distance between the infiltration facility and the location of potential impact is less than the distances provided in Table 7, more detailed mounding assessment is required. The distances in Table 7 are not based on a detailed technical analysis but a consensus of professionals that conduct detailed groundwater mounding assessments.

Estimating the change in groundwater flux associated with the project requires estimating the amount of additional groundwater recharge due to the project and the amount of groundwater flux upgradient of the location of potential impact. The amount of additional recharge due to the project can be estimated using the following formula:

$$FLUX_p = P\,(IMP_{inf} + 0.5\,VEG_{inf} - 0.5\,VEG_{off}) \tag{12}$$

Where:

$FLUX_p$ = additional groundwater recharge due to project

P = annual precipitation

IMP_{inf} = net impervious portion of catchment converted from off-site discharge to infiltration

VEG_{inf} = net vegetated portion of catchment converted to impervious surface with infiltration

VEG_{off} = net vegetated portion of catchment converted to impervious surface with off-site discharge.

This formula is a very approximate water balance calculation that assumes the following:

1) None of the precipitation on impervious surfaces with off-site discharge recharges groundwater.
2) All of the precipitation on impervious surfaces contributing to infiltration facilities recharges groundwater.
3) 50% of the precipitation that falls on vegetated areas recharges groundwater, the remainder is either evapotranspired or runs off as surface water.

Groundwater flux at the location of potential impact is estimated to equal the total groundwater recharge in the catchment area upgradient of the location of potential impact. It is calculated using the following very approximate water balance equation:

$$FLUX_{bg} = P \times CATCH_{\mathrm{w}} \times CATCH_{\mathrm{l}} \times PER\% \tag{13}$$

Where:

$FLUX_{bg}$ = pre-project groundwater flux at the location of potential impact

$CATCH_w$ = width of infiltration facility (or multiple facilities) perpendicular to the groundwater flow direction

$CATCH_l$ = length of catchment area from the location of potential impact to the top of the groundwater basin and parallel to the groundwater flow direction

$PER\%$ = percent pervious area in the catchment area upgradient of the location of potential impact.

The equation assumes that the width of the catchment area is 2 × $CATCH_w$ and 50% of the precipitation on pervious surfaces within the catchment recharges groundwater (these factors cancel out in the equation). $PER_{\%}$ can be estimated based on land use as following:

1) $PER_{\%}$ = 0.1 for industrial/commercial land use
2) $PER_{\%}$ = 0.5 for single family residential land use
3) $PER_{\%}$ = 0.9 for rural/undeveloped land use

The percent increase in groundwater recharge ($FLUX_\Delta$) is calculated as follows:

$$FLUX_\Delta = \frac{FLUX_p}{FLUX_{bg}} \tag{14}$$

It is unlikely that groundwater mounding will cause adverse impacts and more detailed assessment is not necessary if $FLUX_\Delta$ < 5% and the distance between the location of potential impact and the infiltration facility are greater than the values provided in Table 7.

Table 8 Screening-level criteria to determine if detailed groundwater mounding assessment is required.

Contributing Impervious Area	Distance to Location of Potential Impact	$FLUX_\Delta$
<1	>200	<5%
1-5	>400	<5%
5-10	>800	<5%
>10	>1,500	<5%

Notes: Detailed groundwater mounding assessment is not required if both the distance criteria and the $FLUX_\Delta$ criteria are true.

7.2 Detailed Groundwater Mounding Assessments

The methodology and scope for a detailed groundwater mounding assessment should be tailored to address the concerns identified during the screening assessment. It is the responsibility of a qualified hydrogeologist to determine the appropriate methodology and scope for the assessment and then implement or supervise the work. The primary elements of a detailed groundwater mounding assessment include:

1) Collection of sufficient subsurface information to develop a conceptual model of subsurface stratigraphy and hydrogeology.
2) Hydrologic modeling of stormwater discharge to the infiltration facility.
3) Development of a computer model that represents the characteristics of the infiltration facility, the subsurface stratigraphy/hydrogeology, and the routing of stormwater discharge into the infiltration facility.
4) Evaluating the potential impacts of groundwater mounding.

Each of these elements are discussed in the following sections.

7.2.1 Development of Conceptual Model

The conceptual model includes the following elements:

1) The location, dimensions, and construction details of the proposed infiltration facility (or multiple facilities).
2) The thickness and K_s of significant stratigraphic layers within the area of interest down to the uppermost perching layer beneath the facility. If the uppermost aquifer beneath the facility is relatively thick (>10 ft) it may not be necessary to define the full thickness of the aquifer.
3) Fixed head or no-flow boundary conditions that define the horizontal extent of the simulated region and the regional groundwater hydraulic gradient. If these boundary conditions cannot be defined using physical features (surface water bodies, documented seepage, geologic mapping, etc.) within a reasonably close proximity of the infiltration facility, it may be necessary to define artificial boundary conditions that replicate what is observed in the field and won't appreciably affect the results of the numerical model.

4) Documenting the variation in groundwater elevations due to seasonal variability.

5) For alluvial settings, documenting water level change in nearby surface water bodies. Tidal fluctuations and river flood elevations can influence groundwater elevations for alluvial aquifers.

In some cases, there may be sufficient information collected during the feasibility assessment, infiltration testing, and the groundwater mounding screening assessment to develop the conceptual model. If the need for detailed groundwater mounding assessment is identified early in project planning, these activities can be modified to assist in development of the conceptual model. For example, one of the best sources of hydrogeologic understanding is to monitor groundwater elevations beneath infiltration test facilities during infiltration testing. This information provides excellent calibration data for development of the numerical model.

It may be necessary to collect additional information to support development of the conceptual model. This information could include borings, groundwater monitoring and/or test wells, and aquifer testing. The level of additional data collection should be designed to meet the needs of the mounding assessment.

7.2.2 Hydrologic Modeling of Stormwater Discharge

The groundwater mounding assessment requires a time series of stormwater discharge into the infiltration facility (or multiple facilities) to estimate the resulting groundwater conditions. The stormwater engineer is generally responsible for conducting hydrologic modeling of the proposed stormwater management system to generate the time series. The time series generated by the hydrologic models often include many decades and modeling the entire series for the groundwater mounding assessment is unnecessary. The goal is to select a period of heavy precipitation that would be representative of the worst-case scenario with regards to groundwater mounding and associated impacts. If the concern is limiting the capacity of the infiltration facility, the worst-case scenario may be the highest 1-day precipitation following a week of heavy rain or the highest 1-hour precipitation during one of the highest 1-day precipitation event. For groundwater impacts far from the infiltration facility (e.g., flooding or an increase in landslide risk) the worst-case scenario may be the highest one-month precipitation during a relatively wet rainy season, which may require simulating the entire wet season.

The qualified hydrogeologist should work with the stormwater engineer to identify one or more worst-case scenarios to evaluate using the numerical groundwater model. The stormwater modeling engineer can produce a time series of runoff volumes/rates in a format that can be used by the qualified hydrogeologist. Generally, 15-minute timesteps are not necessary for groundwater modeling. One-hour timesteps are suitable for near-field impacts and simulations lasting less than a week. Daily timesteps are suitable for far-field impacts and simulations lasting more than a month.

7.2.3 Development of the Numerical Model

Numerical models can range from the very simple to very complex. The level of complexity should be commensurate with the needs of the groundwater mounding assessment, the size of the project, and the hydrogeology.

A relatively simple axisymmetric numerical model is sufficient if the groundwater table is relatively flat, the infiltration facility can be approximated as a circular facility, and the purpose is to determine if groundwater mounding will limit the capacity of the facility or determine the extent of groundwater mounding near a single infiltration facility. These models assume that the regional hydraulic gradient does not measurably impact the results. These situations can be simulated using a layered unsaturated/saturated model, as demonstrated by Kindred (2022. Volume VI).

A relatively simple two-dimensional (2-D) plan-view numerical model may be suitable for a variety of scenarios. Some scenarios that are best simulated using a 2-D plan-view model include: 1) a long narrow infiltration facility that can’t be simulated using an axisymmetric domain, 2) a site with multiple infiltration facilities that contribute to the same mound, , and 3) the potential for far-field impacts where the regional hydraulic gradient may play an important role in the simulation. Generally, these models only simulate saturated flow and the infiltrating stormwater is applied at the water table (i.e., unsaturated flow between the bottom of the infiltration facility and the water table is not modeled)

A full three-dimensional plan-view model may occasionally be warranted for large projects with the potential for significant adverse impacts and complex hydrogeologic conditions. These models generally require a significant investment in drilling, well installation, testing, and groundwater monitoring to refine the conceptual model.

7.2.4 Impact Assessment

The groundwater mounding assessment should provide a projection of the maximum groundwater elevation that will occur at the facility and locations of potential impact as a result of infiltrating the worst-case runoff event(s) for the facility as designed. The next step is to determine if the groundwater mounding with either limit the capacity of the proposed infiltration facility or result in unacceptable impacts.

If the groundwater mounding assessment indicates that groundwater mounding will limit the capacity of the proposed infiltration facility, the qualified hydrogeologist shall work with the stormwater design engineers to modify the stormwater site plan to address the groundwater mounding limitations. In some cases, these limitations can be addressed by moving the infiltration facility to another location, replacing a single infiltration facility with multiple smaller and dispersed facilities, or increasing the storage capacity of the system. The groundwater model can be used to simulate these design alternatives and identify an approach that meets the flow control objectives.

The groundwater model can be used to evaluate the water table rise near below-grade structures, areas of surface flooding and landslide hazards. Potential impacts to landslide hazards shall be evaluated by a qualified geotechnical engineer based on the groundwater mounding results. In some cases, the mounding impacts can be reduced by moving the infiltration facility to another location or replacing a single infiltration facility with multiple smaller and dispersed facilities. However, in some cases, the mounding analysis may demonstrate that stormwater infiltration will cause unacceptable adverse impacts and is not feasible.

7.3 Reporting

The results of the groundwater mounding assessment can be included in the Infiltration Report, described in Section 5.7. Alternatively, the results of the groundwater mounding assessment can be provided in a stand-alone report.

8 Definitions

α*: Soil sorption number (1/ft), which represents the capillarity or suction capacity of the soil and can be visualized as the inverse of the wetting front height above the water table.

$AREA_{ave}$: The ponded area of the facility (ft^2) when $H = 0.5H_{max}$.

$AREA_{max}$: The ponded area of the facility (ft^2) when $H = H_{max}$.

Axisymmetric numerical model: Two-dimensional, cross-sectional model that assumes radial flow from the center. Can be used to simulate infiltration from a single facility that can be represented as an equivalent circle.

CSSBP: Cased steady-state borehole permeameter method.

C_c: Cased shape factor (dimensionless), which represents the geometry of the infiltration test facility in the CSSBP equation.

CF_c: Clogging correction factor, used to calculate K_d from K_b.

CF_f: Flow correction factor, used to calculate K_d from K_b.

CF_m: Mounding correction factor, used to calculate K_d from K_b.

CF_r: Recharge correction factor, used to calculate K_d from K_b.

CF_u: Uncertainty correction factor, used to calculate K_d from K_b.

CF_w: Well correction factor, used to calculate K_d from K_b.

C_u: Uncased shape factor (dimensionless), which represents the geometry of the infiltration test facility in the USSBP equation.

D_{max}: Longest horizontal ponding dimension of the infiltration facility (ft).

Deep infiltration: Infiltration from a facility that extends more than 10 ft below final grade.

Drywell: A dug or drilled infiltration facility where $r_e < H_{max}$ of the facility and capacity is dominated by horizontal flow out the sides of the facility.

Filter pack: Coarse sand or gravel placed in the annular space around the well screen to create a good connection with the native soils and prevent caving of the borehole walls.

$FLUX_{\Delta}$: The percent increase in groundwater recharge due to an infiltration facility compared with background groundwater flux. Calculated using: $FLUX_{\Delta} = FLUX_p/FLUX_{bg}$.

$FLUX_p$: Groundwater recharge due to an infiltration facility.

$FLUX_{bg}$: Background groundwater flux.

H: Ponding depth (ft).

H_{max}: The average ponding depth (ft) when the facility is at its maximum capacity

Horizontal infiltration facility: An infiltration facility where $r_e > H_{max}$ and infiltration capacity is dominated by vertical flow out the bottom of the facility. Generally, infiltration ponds, bioretention facilities, and permeable pavements are horizontal infiltration facilities.

Infiltration Interval: The infiltration interval occurs between the steady-state ponding depth or the top of the filter pack (whichever is less) and the bottom of the facility (either the base of an open excavation or the bottom of the filter pack for a test well).

K_b: Bulk hydraulic conductivity (ft/d) is the measurement of permeability based on an infiltration test that is evaluated using the borehole permeability method. It includes the effects of layering and groundwater mounding which may influence steady state flows during the test.

K_d: Design hydraulic conductivity (ft/d) is the measurement of permeability used to predict the capacity of an infiltration facility. It is calculated by multiplying K_b by correction factors.

K_s: Saturated hydraulic conductivity (ft/d) is the measurement of permeability for a homogeneous, isotropic soil.

MGS Flood: Hydrologic Model used to simulate stormwater flows.

Plan-view numerical model: Two- or three-dimensional horizontal model that can be used to simulate multiple infiltration facilities, complex boundary conditions, and a sloping water table.

Pressure transducer: Electronic device used to measure the water pressure in a column of water, usually expressed as feet of water.

Q: Flow rate (ft^3/d).

Qualified geotechnical professional: Professional engineer or licensed geologist with at least 5 years of experience conducting slope stability assessments (experience must include a minimum of 10 sites).

Qualified hydrogeologist: Licensed hydrogeologist with at least 5 years of experience in groundwater assessments, including groundwater modeling (experience must include a minimum of 10 sites).

Qualified infiltration professional: Professional engineer or licensed geologist with at least five years of experience conducting infiltration assessments (experience must include a minimum of 10 sites).

r_e: Equivalent radius (ft). For circular boreholes, test wells and drywells, r_e equals the radius of the borehole. For excavated test pits and full-scale infiltration facilities, $r_e = \sqrt{AREA/\pi}$ where $AREA$ is the surface area of the pond in the facility. For a rectangular facility, $r_e = \sqrt{Length \times Width / \pi}$.

Seasonal high groundwater: The highest saturated zone below the target infiltration interval for the infiltration facility. For shallow infiltration facilities, this is the saturated zone closest to the ground surface. For deep drywells that have a surface seal, this is the uppermost saturated zone below the filter pack interval.

Shallow infiltration: Infiltration from a facility that is less than 10 ft below final grade.

SWMMWW: 2019 Stormwater Management Manual for Western Washington.

UIC: Underground injection control well, which is any infiltration facility that is deeper than its largest horizontal dimension. A detailed definition is provided in Section I-4.15 (Deep UIC Wells) of the SWMMWW.

USCS: Unified Soil Classification System.

USSBP: Uncased steady-state borehole permeameter method.

WWHM: Western Washington Hydrologic Model used to simulate stormwater flows.

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