EVALUATION OF INFILTRATION TESTING METHODS

Scientific Study

Prepared for: **Safe Clear Water Program**

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Contents

Abstract

Infiltration testing was conducted at three sites in Los Angeles County in 48-inch diameter full-scale drywells and small-diameter test well drilled using both hollow-stem auger (HSA) and Sonic drilling methods. The results were analyzed using the steady-state borehole permeameter (SSBP) method which provides an estimate of bulk hydraulic conductivity (K_b) for the tested interval. The SSBP method can be used to predict the capacity of full-scale drywells with different well diameters and ponding depths using the K_b estimates provided by infiltration testing in small-diameter test wells.

This study provides the following recommendations:

- Sonic wells do not appear to be clogged, although the limited examples provided in this study suggest Sonic wells may under-predict the capacity of full-scale drywells.
- HSA wells are subject to clogging and well development does not remove the clogging. HSA wells are not suitable for predicting the performance of full-scale drywells.
- Small-diameter Sonic test wells constructed with 2-inch diameter well screen may experience significant head loss in the screen and sandpack at high flow rates (greater than ~100 gpm with 10 ft of screen). As a result, these wells may provide K_b estimates that under-predict the capacity of full-scale drywells, which are typically constructed using 6-inch diameter well screen. This issue can be reduced by using 4-inch diameter well screen in the test wells.
- In order to minimize air-entrainment, drop pipe that extends below the water level during the test is required when testing in 2-inch diameter casing. It is also recommended when testing in 4-inch diameter casing but is not necessary when the well is constructed using 6-inch diameter casing and screen.
- Because full-scale drywell capacity can be significantly greater than hydrant capacity, it is important to maximize the flow rate when testing small-diameter test wells in permeable soils. Hydrant flow rate can be maximized by using 2-inch diameter drop pipe rather than smaller diameter drop pipe and removing the filter, meter, and valve assembly at the well head. 2-inch diameter drop pipe requires at least 4-inch diameter well casing.
- The rate of falling head after the water is turned off can provide important information regarding the degree of perching and groundwater mounding. If the test well drains slowly relative to the steady-state test capacity, then more conservative correction factors should be used for estimating the capacity of full-scale drywells.

Although not an issue in this study, experience at other sites and discussions with Torrent Resources indicates that caving is a significant issue when drilling full-scale drywells using a solid-stem auger in clean sands and gravels. In many cases, it is not feasible to drill more than 10 ft into the permeable formation. Fortunately, although these soils may not stand open, they do provide high infiltration capacity. If such soils are encountered when drilling small-diameter test wells, it is recommended to complete the test well in the upper 10 ft of clean sand and gravel. This way, the test results will be useful for estimating the capacity of a full-scale drywell that has caving issues.

1 Introduction

Stormwater infiltration drywells provide significant opportunity to infiltrate stormwater runoff into the subsurface and are becoming a common practice in southern California, both to reduce peak surface flows and recharge groundwater. Los Angeles County Department of Public Works Administrative Manual GS200.1: *Guidelines for Geotechnical Investigation and Reporting, Low Impact Development Stormwater infiltration* (LA County, 2021) provides methods for conducting stormwater infiltration testing and predicting the capacity of full-scale infiltration facilities.

1.1 Limitation of GS200.1 Large-Diameter Boring Methods

There have been unpublished reports that the methods in GS200.1 tend to significantly underpredict actual drywell performance, resulting in more drywells than needed and significantly higher construction and long-term maintenance costs. In other cases, the uncertainty in the measurements resulted in predicted capacities that were higher than actual drywell capacity, resulting in fewer drywells that needed to meet capacity objectives. GS200.1 recommends infiltration testing in a large-diameter boring with a diameter of 18-36 inches for estimating the capacity of drywells. Either a falling-head or constant-head test may be performed, depending on the falling head rate after a 1-hour presoak period.

There are several challenges with the large-diameter boring method:

- Installing an 18-inch diameter test well is expensive and similar in cost to a full-scale drywell (generally 48 inches in diameter). Project owners are reluctant to spend the money to conduct the tests.
- The maximum flow from a fire hydrant ranges from 140 to 230 gpm, depending on the fire hydrant, meter and valve arrangement, length of fire hose, and the length of drop pipe. In permeable soils, this flow rate may not be sufficient to match the maximum ponding depth of the full-scale drywell.
- The falling-head large-diameter boring (FHLD) approach includes the following requirement: "The field infiltration rate must account for non-vertical flow through the sides of the boring in addition to the bottom of the boring." The approach to comply with this requirement is not provided and it is unclear how the rate of falling-head can be used to estimate the capacity of a full-scale drywell.
- The constant-head large-diameter boring (CHLD) methods states that: "The infiltration rate can be determined by dividing the average stabilized volumetric rate by the total surface area of infiltration within the boring." Unfortunately, the CHLD method does not account for the full dynamics of unsaturated flow, including pressure head (i.e., flow varies as a function of ponding depth in the well) and capillarity (i.e, flow varies as a function of pore space suction).
- The FHLD and CHLD methods do not indicate how the measured infiltration rate shall be used to estimate the capacity of the full-scale drywell, although presumably the intention is to multiply the total surface area of the drywell by the design infiltration rate.
- THE FHLD and CHLD methods do not account for scenarios where the water level rises up into solid casing above the screened interval. This is a significant issue since most drywells are sealed to a depth of 10 ft or more and maximum capacity will be achieved when H approaches the ground surface.

As described in Appendix A, numerical simulations were conducted to replicate both the FHLD and CHLD methods and compare the results with the actual capacity of a full-scale drywell in the same soils. Simulations were conducted for three soil types and the analysis assumed homogeneous and isotropic soils. This analysis demonstrated that neither the FHLD or CHLD methods are very accurate. For the soils and scenarios tested in this analysis, errors ranged from -40% to +64%. The fundamental issue with the GS200.1 methods is that they provide an estimate of infiltration rate, which is intended to represent vertical flow from a shallow large-diameter infiltration facility and is not well suited for predicting horizontal flow from a drywell.

1.2 Project Goals

Given the issues with the current drywell testing methods, the Safe Clean Water program funded this study to evaluate and demonstrate improved infiltration testing methods for stormwater drywells. The goals for the project are to:

1. Identify and evaluate infiltration testing methods that are suitable for estimating drywell capacity over a range of well diameters.
2. Evaluate drilling, well construction, and well-development methods to minimize smearing of fine-grained soil on the borehole walls and other borehole effects.
3. Evaluate and refine testing methods to ensure suitable accuracy while minimizing test complexity and cost.
4. Ensure that the methods provided by this work are reviewed and accepted by stakeholders, including regulators, municipal stormwater managers, and geotechnical/hydrogeologic consultants that regularly conduct infiltration testing and design.

1.3 Review of Stormwater Infiltration Testing Practices and Previous Studies

Appendix B provides a literature review of infiltration testing methods across the United States, including the methods provided in GS200.1 (LA County, 2021). Other than the FHLD and CHLD methods in GS200.1, the literature review did not identify any other methods for testing and estimating the capacity of stormwater drywells that are currently in use by stormwater permitting agencies.

The United States Environmental Protection Agency (EPA) funded a recent study to investigate borehole permeameter methods and determine if these methods could be adapted for stormwater infiltration testing and design. The study was managed by the Washington State Department of Ecology (Ecology) and conducted by the City of Tacoma with a consultant team. The study resulted in preparation of an infiltration guide that proposed methods for infiltration testing and designing infiltration facilities. The full technical report and the infiltration guide are available at: https://kindredhydro.com/infiltration-downloads-1. The Ecology study included both numerical analysis and field testing and evaluated methods for both shallow and deep infiltration facilities, including drywells. Field work for the Ecology study included both shallow and deep infiltration testing. Shallow field testing was conducted at four sites with different soils at each site. Each shallow test site includes testing in two shallow test pits and three shallow test wells (less than 10.5 ft deep). Deep well testing was conducted at eight sites in deep test wells that were 6-8 inches in diameter and 21 to 87 ft deep.

As discussed in Appendix A, infiltration methods that provide an estimate of infiltration rate do not account for the full dynamics of unsaturated flow from a borehole and are not well suited for estimating drywell capacity. For this reason, the Ecology study focused on methods that account for the full dynamics of unsaturated flow, including falling-head borehole permeameter (FHBP) methods and steady-state borehole permeameter methods (SSBP). A history of the borehole permeameter methods is provided by Kindred and Reynolds (2020).

The FHBP method relies on a test well with a screened interval and solid casing above the screen interval. It is significantly different than the FHLD method in GS200.1 because it assumes instantaneous filling of the test well above the screened interval and tracking the rate of fall until the water level falls into the screened interval. The method requires knowing the porosity and pre-test water content of the formation, both of which are difficult to measure accurately. The test cannot be repeated until the formation has fully drained and returned to ambient water content in the vicinity of the test well, so it is important to conduct the test correctly the first time. Based on numerical simulations that could specify soils parameters and simulate instantaneous filling of the test well, the FHBP method is theoretically relatively accurate. However, the results were not accurate if instantaneous filling of the well was not achieved and numerical simulations using reasonable methods of adding water to the well demonstrated that the FHBP method was only accurate for relatively low-permeability soils and small test wells. In addition, the FHBP method would only saturate a small volume of soil around the well and it is highly likely that this volume of soil would be disturbed during installation of the test well. Based on these considerations, the FHBP method was not recommended for inclusion in the Infiltration Guide.

The SSBP method is performed using a similar field approach as the CHLD method in GS200.1 except there is no distinct pre-soak period followed by a falling-head period. Instead, steady-state conditions are simply maintained for a period of 6 hours. Analysis of the SSBP field results relies on simple arithmetic equations that account for the full dynamics of unsaturated flow from a borehole. These equations are provided in Appendix C. The SSBP method is suitable for uncased scenarios (USSBP), when hydraulic head (H) < sandpack interval (L), and cased scenarios (CSSBP) when $H > L$. The sandpack interval is the portion of the borehole that is filled with permeable backfill to connect the well screen with the formation. It is usually a longer interval than the screened interval. Based on numerical calibration of the fitting parameters, the SSBP methods have an accuracy of ± 13% for homogeneous and isotropic soils and are calibrated for H/r_b ratios ranging from 0.05 to 200 (r_b is the borehole radius) and L/r_b ratios ranging from 4 to 100. For the examples provided in Appendix A, the SSBP methods provided accurate estimates of drywell capacity, with errors ranging from -4% to +3%. This is significantly better than the FHLD and CHLD methods in GS200.1.

The SSBP methods assume homogenous and isotropic soil media and the Ecology study limits use of the term saturated hydraulic conductivity (K_s) for homogeneous and isotropic soils. Numerical calibration of the SSBP methods and the simulations provided in Appendix A were conducted using homogenous and isotropic soils and the results were reported using K_s. Since real-world soils are almost always layered, the Ecology study used the term "bulk hydraulic conductivity" (K_b) to report the results for field tests or numerical simulations of layered soils. This study will also use K_b to report the results of field tests and numerical simulations of field tests.

Some of the lessons learned from the Ecology study regarding drywell testing and design include:

- With few exceptions, infiltration test wells drilled using Sonic methods do not appear to be clogged. Based on a single example, test wells drilled using hollow stem auger (HSA) methods appear to be clogged and do not provide a reliable estimate of K_b.
- No well development was conducted for these tests and it is unknown if HSA wells can be remediated with well development.
- Free-falling water in well casing results in air-entrainment and reduces well capacity. The effect is greater in 2-inch diameter casing than 4-inch diameter casing and increases as the flow rate increases. Drop pipe that discharges below the water level during the test eliminates air entrainment.
- Velocity head can increase head below the bottom of the drop pipe. This resulted in transducer readings in the bottom of well that were higher than the ground surface, even when the water level observed at the top of the well is below the ground surface. The effect was only observed in 2-inch diameter casing, and not the 4-inch diameter casing.

1.4 Scope of Work

The scope of work for this project included the following tasks:

- Review test methods across the U.S.
- Conduct deep infiltration testing at three sites.
- Compare small-diameter test well results with full-scale drywell results.
- Summarize lessons learned and write report.
- Communicate results and recommendations to the stormwater community.

1.5 Acknowledgements

Funding for this study was provided by the Safe Clear Water Program – Measure W, as authorized by the Upper Los Angeles River (ULAR) Watershed. Access to public land and logistical support for the field work was provided by the Los Angeles County Public Works department, the Los Angeles County Department of Park and Recreation, and the City of Glendale.

2 Materials and Methods

2.1 Test Sites and Well Installation

Infiltration testing was conducted at the following three sites:

1. **Mary Bethune Park** south of the City of Los Angeles. One drywell and two test wells were installed and tested at this site.
2. **Glendale Site 1**: Testing was conducted in an existing drywell installed by the City of Glendale in October 2022. The intention was to test two test wells at this site but the drillers determined that underground utility lines were too close together to install wells, so testing was limited to the existing drywell.
3. **Glendale Site 2:** Testing was conducted in an existing drywell installed by the City of Glendale in October 2022 and in two test wells installed for this project.

Locations of the three test sites are shown in Fig. 1. Grainsize analyses were conducted on selected soils samples and the results are provided in Appendix D. Boring logs for the wells installed during the project are provided in Appendix E and well construction details are summarized in Table 1.

Table 1: Summary of Drywell and Test Wells.

Test Name	Borehole Depth (ft)	Drilling Method	Install Date	Boring Dia. (in)	Casing Dia. (in)	Screen Type	Filter Pack Interval (ft)
Bethune Site							
B-Dry	60	Auger	Apr-2023	48	6	slotted	48-60
B-HSA	59	HSA	Apr-2023	8	3	perf. with fabric	46.5-59
B-Sonic	70	Sonic	Apr-2023	8	2	slotted	48-60
Glendale Site 1							
G1-Dry	45	Auger	Oct-2022	48	6	slotted	14-45
Glendale Site 2							
G2-Dry	45	Auger	Oct-2022	48	6	slotted	13-45
G2 HSA	45	HSA	Jun-2023	8	2	slotted	12-45
G2-Sonic	55	Sonic	Jun-2023	8	2	slotted	12.5-44.5

Notes: B – Bethune; G1 – Glendale Site 1; G2 – Glendale Site 2; Sonic – Sonic drilling rig; HSA – Hollow stem auger drilling rig; Dry - Drywell

2.1.1 *Mary Bethune Park Site*

Three wells were installed at the Bethune site during April 2023. The wells were approximately 30 ft apart and the site layout is shown on Fig. 2. Each of the wells is described below:

1. **B-Dry**: This well was installed by Torrent Resources out of Bloomington, CA as a typical drywell minus the concrete surface liner. It was installed using a 48-inch diameter solid stem auger drilling rig and the borehole was drilled to a depth of 60 ft. The soils were described in the field and soil samples were collected for grainsize analysis. The well was constructed with 10 ft of 6-inch diameter slotted screen and 50 ft of solid 6-inch diameter casing. Washed pea gravel was placed in the annular space from 48-60 ft and the remainder of the annular space was backfilled with bentonite slurry grout.

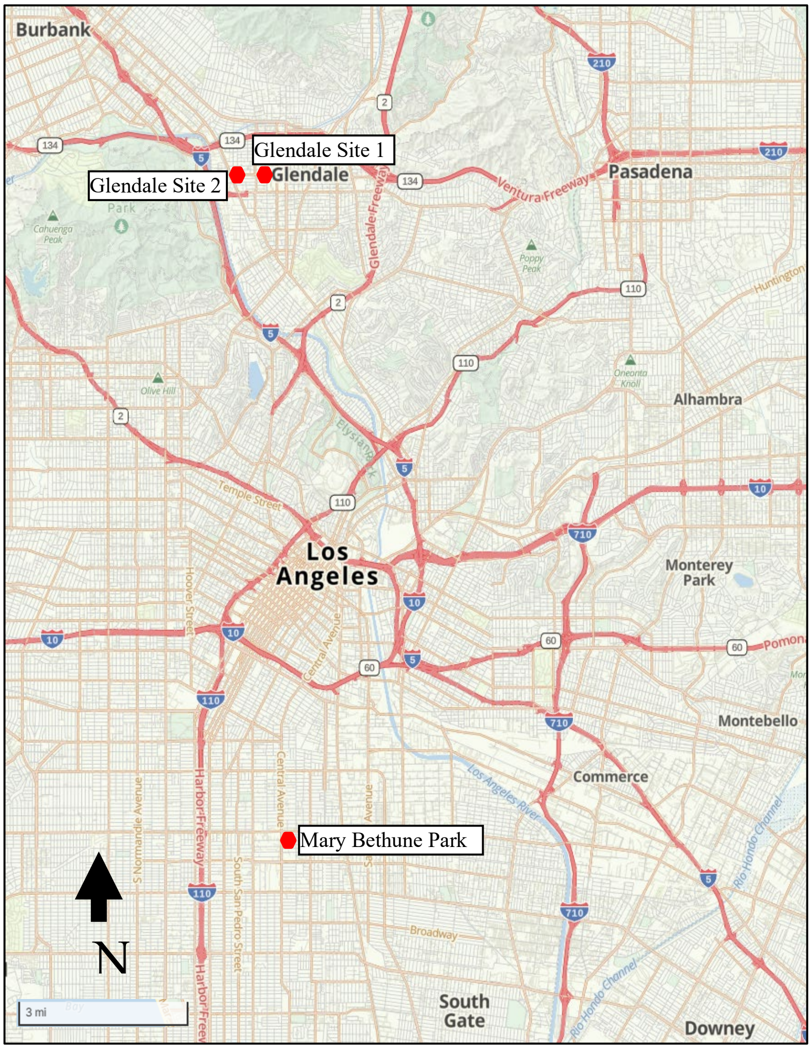
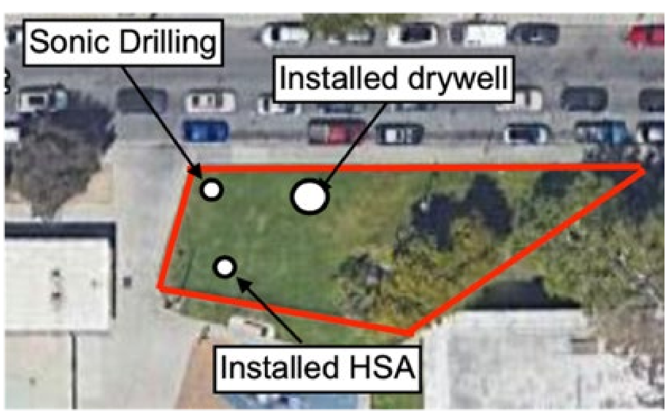


Fig. 1: Vicinity map showing the location of the three test sites.

Fig. 2: Mary Bethune Park Site Layout



2. **B-HSA:** This well was installed by Cascade Environmental out of Upland, CA using an 8-inch diameter hollow-stem auger (HSA) drilling rig. It was drilled to a depth of 59 ft. The soils were described in the field and soil samples were collected for grainsize analysis. The well was constructed with 10 ft of 3-inch diameter perforated pipe wrapped in fabric and 50 ft of solid 3-inch diameter casing. The drillers attempted to place washed crushed gravel around the perforated pipe but most of the hole caved before the gravel could be placed. Washed crushed gravel was placed from a depth of 46.5-50 ft. The remainder of the annular space was backfilled with bentonite chips that were hydrated as they were placed.

3. **B-Sonic**: This well was installed by Cascade Environmental using a sonic drilling rig and a 8-inch diameter core barrel. It was drilled to a depth of 70 ft and the bottom of the boring was backfilled with bentonite chips to a depth of 60 ft. Soil samples were collected from 50 to 70 ft for grainsize analysis. The well was constructed with 10 ft of 2-inch diameter slotted screen and 50 ft of solid 2-inch diameter casing. Washed sand was placed in the annular space from a depth of 48-60 ft. The remainder of the annular space was backfilled with bentonite chips that were hydrated as they were placed.

When testing was complete the wells were abandoned by removing the upper 5-10 ft of casing and backfilling the well with bentonite grout using a tremie pipe from the bottom of the screen.

2.1.2 Glendale Site 1

The City of Glendale recently installed a drywell at this site and the intention was to install two test wells and then conduct infiltration testing on all three wells. When the drillers arrived to install the test wells they determined that the utilities were too close together to safely install the wells. Therefore, only the drywell was tested. The well completion details are described below:

1. **G1-Dry**: This drywell was installed by Torrent Resources in October of 2022 and was placed into service at the end of 2022. It was drilled using a 60-inch diameter solid stem auger to a depth of 14 ft and a 48-inch diameter solid stem auger to a depth of 45 ft. The well was constructed with 10 ft of 6-inch diameter slotted screen and approximately 30 ft of solid 6-inch diameter casing. Washed rock was placed in the annular space from depth of 14-45 ft. A 54-inch diameter concrete liner was placed on the gravel and the remainder of the annular space was backfilled with bentonite slurry grout.

The drywell was left in service after testing.

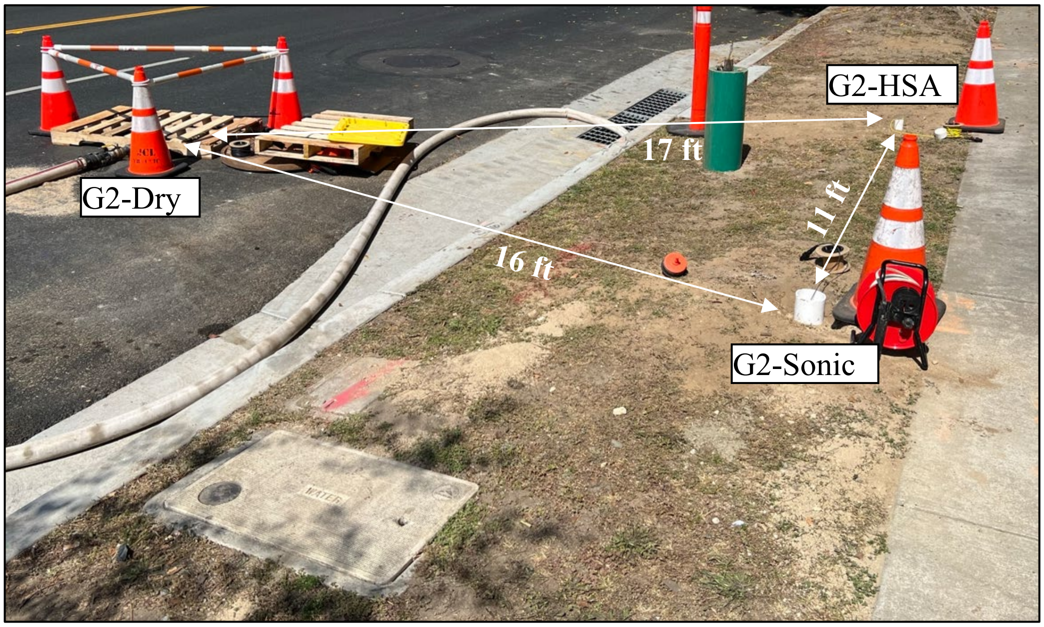
2.1.3 Glendale Site 2

The City of Glendale recently installed a drywell at this site and two test wells were installed nearby for this project. The layout of the site is shown on Fig. 3 and each of the wells is described below:

1. **G2-Dry**: This drywell was installed by Torrent Resources in the same manner as G1-Dry.

2. **G2-HSA**: This well was installed by Cascade Environmental in June of 2023 using an 8-inch diameter hollow-stem auger (HSA) drilling rig. It was drilled to a depth of 45 ft. The soils were described in the field and soil samples were collected for grainsize analysis. The well was constructed with 30 ft of 2-inch diameter slotted screen and 15 ft of solid 2-inch diameter casing. Washed sand was placed in the annular space from a depth of 12-45 ft. The remainder of the annular space was backfilled with bentonite chips that were hydrated as they were placed.

Fig. 3: Glendale Site 2 Layout



3. **G2-Sonic**: This well was installed by Cascade Environmental in June 2023 using a sonic drilling rig and a 8-inch diameter core barrel. It was drilled to a depth of 55 ft and the bottom of the boring was backfilled with bentonite chips to a depth of 44.5 ft. The soils were described in the field and soil samples were collected for grainsize analysis. The well was constructed with 30 ft of 2-inch diameter slotted screen and 15 ft of solid 2-inch diameter casing. Washed sand was placed in the annular space from a depth of 12.5-44.5 ft. The remainder of the annular space was backfilled with bentonite chips that were hydrated as they were placed.

When testing was complete the two test wells were abandoned by removing the upper 5-10 ft of casing and backfilling the well with bentonite grout using a tremie pipe from the bottom of the screen.

2.2 Test Procedures

Testing at the SSBP tests were conducted in each well using the following general procedures:

1) Well development was conducted in the test wells by alternating between adding water and pumping water out of the well using a small submersible pump. The pump was used as a surge block by lifting it up and down throughout the entire well screen. Water would flow into the formation when water was added and sediment-ladened water would flow back into the well during the pumping phase. The sonic wells cleaned up quickly and the HSA wells took longer to clean up.
2) After development, steady-state tests of approximately 6 - 8 hours were conducted in all the wells.
3) A pressure transducer was placed in the bottom of the well and was set to record the water depth once per minute. The pressure transducer was connected to a data cable that allowed real-time monitoring of the depth of water during the test.
4) All head elevations used in the analysis are referenced from the bottom of sandpack, based on the assumption that water flows freely through the sandpack with minimal head loss. Therefore, the depth of sandpack beneath the screen was added to the transducer reading in the bottom of the well for graphing and analysis.

5) Water from a fire hydrant was discharged into the well at a rate that maintained the water level (based on the pressure transducer in the bottom of the well) at or near a steady-state elevation.
6) In some cases, drop pipe was attached to the flowmeter assembly to convey the water into the well casing and discharge below the steady-state water level during the test.
7) Flow rates were measured using flowmeters and recorded at regular intervals during the tests.
8) Water levels were recorded at regular intervals during the tests to determine when it was necessary to change the flow rate to maintain the water level near the top of the sandpack.
9) The transducer was left in the well after the water was turned off to record the water levels during the falling head portion of the test.
10) The results were evaluated using the SSBP methods provided in Appendix C.

3 Results

This section presents the test results for the three sites included in this study.

3.1 Bethune Park Test Site

This section provides the infiltration testing result for the three wells installed at Bethune Park, including a 48-inch diameter drywell, an 8-inch diameter HSA well, and a 6-inch diameter sonic well. The geology at the site generally consisted of very silty sand (SM) to sandy silt (ML) to a depth of 46-47 ft and well-graded gravelly fine-medium sand (SW) with trace silt from 46-70 ft. The drywell and the sonic well were completed with sandpack from 48-60 ft and the HSA well was constructed with sandpack from 46.5-59 ft. The sonic well was drilled to a depth of 70 ft and there was no perching layer observed below the sandpack interval.

3.1.1 Well B-Dry

Two tests were conducted in B-Dry. The first test was conducted on April 12, 2023. No well development was conducted before testing. As shown on Fig. 4(a), the test reached steady-state within approximately 70 minutes with H = 5.9 ft and Q = 145 gpm. This was the maximum capacity of the hydrant after flowing through approximately 600 ft of fire hose and the meter and valve at the well head. Water was discharged into the 6-inch casing through 2-inch diameter drop pipe that was initially placed to a depth of 50 ft. When it became clear that the water level would not rise above the bottom of the drop pipe, the drop pipe was extended to a depth of 55 ft at 210 minutes. The deeper drop pipe did not significantly change the water level measurements, indicating that air entrainment is not a significant issue when the water is discharged into a 6-inch diameter casing. The water level began to drop after about 250 minutes and by the end of the test at 363 minutes H was 5.7 ft with a Q of 146 gpm. It is not certain why well performance improved with time, but it may be due to well development during the test.

The second test was conducted on April 26, 2023, two weeks after the first test. No well development was conducted. As shown on Fig. 4(b), the test reached steady-state within approximately 50 minutes. No drop pipe was used during this test. The water level did not change significantly after the first 50 minutes of the test and at the end of the test (354 minutes) H was 6.1 ft with a Q of 142 gpm.

The water level shown on Fig. 4 (a and b) was measured using a transducer in a 1-inch piezometer placed in the outer edge of the gravel pack to minimize turbulence. During the first test a transducer was also placed in the bottom of the 6-inch diameter casing for comparison with the water level in the gravel pack and estimate the head loss through the screen and gravel pack. The head loss between the 6-inch casing and the edge of the gravel pack is shown in Fig. 4(c). For the first 6 min. of the test, the water level in the gravel pack piezometer was actually higher than the water level in the 6-inch casing. It is surmised that this may be due to air entrainment in the casing which would mean that the average water level in the casing would be higher than the transducer reading in the bottom of the casing. From 6 minutes until 210 minutes when the drop pipe was lowered 5 ft, the head loss from inside the 6-inch casing to the edge of the gravel pack was approximately 0.1 ft. Once the drop pipe was lowered to the water level, the head loss increased to approximately 0.17 ft for the remainder of the test. These results indicate that the head loss through 6-ft of screen and gravel pack is nominal at a flow rate of 145 gpm.

The results were analyzed using the methods described in Appendix C and summarized in Table 2. As indicated in the table, the first test provided an estimated K_b value of 135 ft/d and the second test provided an estimated K_b value of 120 ft/d. The reason for the decrease in performance is uncertain but one potential explanation is that the background moisture content was higher in the second test due to residual moisture content from the first test, thus reducing the capillarity of the soil. This explanation is unlikely, however, since removing the capillary term from the SSBP equation would only increase K_b to 125 ft/d.

The falling head portion of the test after the water is turned off can provide a good indication of the extent of perching on low-permeability layers within and below the screened interval of the formation. As shown on Fig. 4(a and b) the well drained quickly once the water was turned off, indicating minimal perching and/or groundwater mounding.

Fig. 4: Test results for drywell B-Dry.

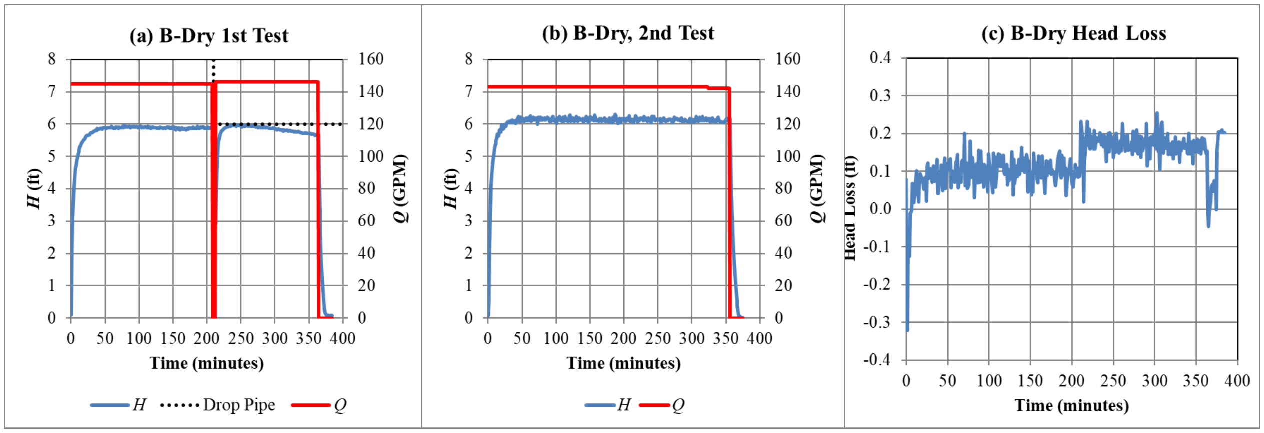


Table 2: Results of SSBP tests in drywell B-Dry.

Parameter	**Units**	**1st Test**	**2nd Test**
Ponding head (H)	ft	5.7	6.1
Flow rate (Q)	gpm	146	142
Borehole radius (r_b)	in.	24	
Well casing radius (r_c)	in.	3	
Saturated sandpack length (L)	ft	5.7	6.1
Assumed sorptive number for med. sand (α*)	1/ft	3.36	
Soil classification		SP	
Calculated hydraulic conductivity (K_b)	**ft/d**	**135**	**120**

3.1.2 Well B-HSA

The test in B-HSA was conducted on April 14, 2023. A short infiltration test was conducted before well development to estimate the benefits of well development. The pre-development test was 80 minutes long and there was 59 ft of head rise at a flow of 12.5 gpm. Well development was conducted for 30 minutes and the water was clear at the end of development.

The infiltration test conducted after well development is shown on Fig. 5. Water was discharged into the well using a 1-inch drop pipe to a depth of 35 ft, well below the water level during the test. The test began at a rate of 21 gpm and the head rose to 34 ft after 52 minutes. At this stage, it was apparent that flow from the well was restricted and would not provide an accurate estimate of K_b. The remainder of the test was conducted by adjusting the flow to maintain the water near the ground surface. The test was close to steady state almost immediately at the higher flow rate and only required a small decrease in the flow rate at 206 minutes. The water was turned off at 306 minutes with H = 59.2 ft and Q = 31.1 gpm.

The results were analyzed using the methods described in Appendix C. As indicated in Table 3, the test provided an estimated K_b value of 5.5 ft/d, 95% less than the K_b measured in the B-Dry, and the results are not considered valid. It was unknown at this stage if the very poor performance of the HSA well was due to clogging or the well completion method (perforated pipe wrapped with fabric rather than slotted pipe and no fabric).

Fig. 5: Test results for test well B-HSA.

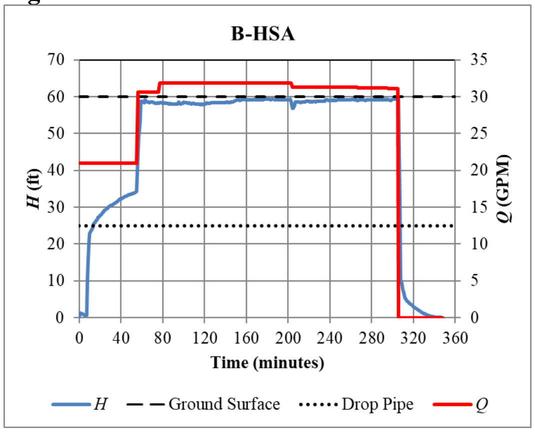


Table 3: Results of SSBP tests in test well B-HSA.

Parameter	**Units**	**306 min.**
Ponding head (H)	ft	59.2
Flow rate (Q)	gpm	31.1
Borehole radius (r_b)	in.	4
Well casing radius (r_c)	in.	1.5
Saturated sandpack length (L)	ft	12.5
Assumed sorptive number for med. sand (α*)	1/ft	3.36
Soil classification		SP
Calculated hydraulic conductivity (K_b)	**ft/d**	**120**

3.1.3 Well B-Sonic

Two tests were conducted in B-Sonic. Well development was conducted for 15 minutes before the first test on April 25, 2023. Although 1,500 gallons of water was pumped into the well during development it was very difficult to maintain water water in the well during pumping due to the fast drainage rate. The water was clear after 15 minutes and well development was terminated.

The first test started with 35 ft of 1-inch diameter drop pipe but the flow was limited to 83 gpm with the fire hydrant fully open. The drop pipe was removed at 190 minutes to reduce the head loss and the flow increased to 117 gpm with the hydrant fully open. As shown on Fig. 6(a), the test never reached steady-state. At 187 minutes, before the drop pipe was removed, H was 12.5 ft with a Q of 83 gpm. At the end of the test (398 minutes) H was 20.7 ft with a Q of 117 gpm.

The second test was conducted on April 27, 2023 and was designed to match the 6 ft of head during the drywell test. Water was discharged into the well through 1-inch diameter drop pipe to a depth of 55 ft. As shown on Fig. 6(b), the test reached steady-state within approximately 200 minutes with Q = 34 gpm. By the end of the test (350 minutes) H was 5.9 ft with a Q of 34 gpm.

The second test was conducted on April 27, 2023, two days after the first test, and is shown on Fig. 6(b).
The results were analyzed using the methods described in Appendix C and summarized in Table 4. As indicated in the table, the first test provided an estimated K_b value of 105 ft/d. The second test was conducted at two different flow rates. After the first 187 minutes, with H = 12.5 ft, the second test provided an estimated K_b value of 80 ft/d. At the end of the test (398 minutes), with H = 20.7 ft, the second test provided an estimated K_b value of 52 ft/d. The reason for the decrease in performance with higher H and Q is uncertain. It may be due to head loss through the well screen and sandpack.

The first test did not reach steady state while the second test achieved steady state after 200 minutes. The likely explanation for this difference is that the zone of saturation was in contact with the overlying fine-grained soils (at H = 13 ft) during the first test and not in contact with the fine-grained soils during the second test. Based on numerical simulations, fine-grained soils generally take longer to reach steady state than coarse-grained soils (Kindred 2022).

Fig. 6: Test results for test well B-Sonic.

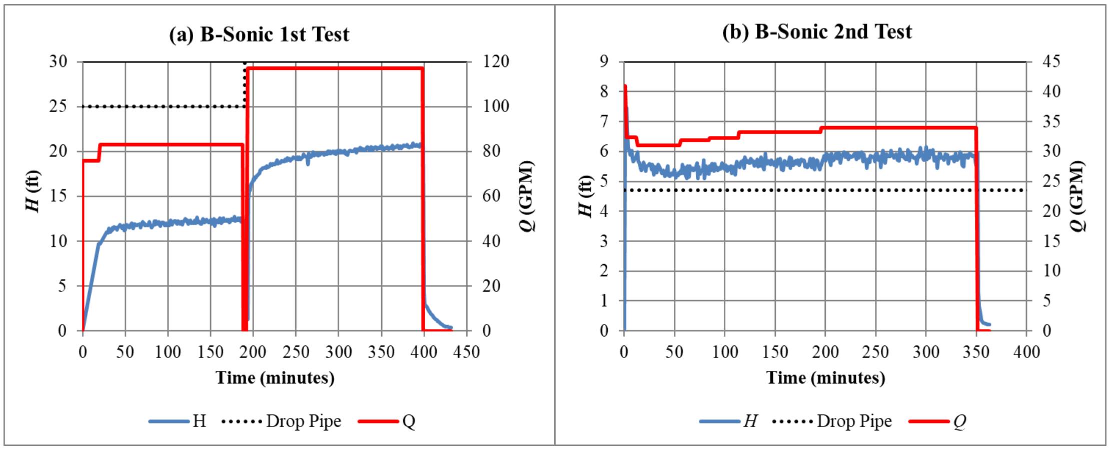


Table 4: Results of SSBP tests in test well B-Sonic.

Parameter	Units	1st Test 187 min.	1st Test 398 min.	2nd Test 350 min.
Ponding head (H)	ft	12.5	20.7	5.9
Flow rate (Q)	gpm	83	117	34
Borehole radius (r_b)	in.	4		
Well casing radius (r_c)	in.	1		
Saturated sandpack length (L)	ft	12	12	5.9
Assumed sorptive number for med. sand (α*)	1/ft	3.36		
Soil classification		SP		
Calculated hydraulic conductivity (K_b)	**ft/d**	**80**	**52**	**105**

3.2 Glendale Site 1

This section provides the infiltration testing result for the 48-inch diameter drywell installed by the City of Glendale at Glendale Site 1 in October of 2022. The geology at the site is expected to be similar to Glendale Site 2, summarized in Section 3.3 The drywell was constructed with sandpack from 14-45 ft and a 56-inch diameter concrete liner sealed with bentonite grout from 0-14 ft.

3.2.1 Well G1-Dry

A single infiltration tests were conducted in G1-Dry on June 2, 2023. No well development was conducted before testing. Water was discharged into the 6-inch casing through 2-inch diameter drop pipe that was placed to a depth of 10 ft and the transducer was placed in the bottom of the 6-inch diameter casing. As shown on Fig. 7, the test began with a target H = 15 ft and was close to steady-state within approximately 30 minutes with a flow of 61 gpm. After 209 minutes the flow rate was increased to the maximum flow allowed by the hydrant (184 gpm). The maximum hydrant flow decreased slightly over the course of the test, likely due to a reduction of water pressure, and was at 174 gpm by the end of the test. The water level rose gradually at this higher flow rate with H = 26.5 ft by the end of the test at 370 minutes.

The results were analyzed using the methods described in Appendix C and summarized in Table 5. As indicated in the table, the initial portion of the test with $H = 15.3$ ft provided an estimated K_b value of 16 ft/d at 209 minutes. The results at 370 minutes provided an estimated K_b value of 20 ft/d. The reason for the increase in performance is uncertain but one potential explanation is that the soils were more permeable above $H = 15$ ft.

In comparison with the Bethune test, the water level declines relatively slowly after the water was turned off. These results indicate significant perching and/or groundwater mounding.

Fig. 7: Test results for drywell G1-Dry.

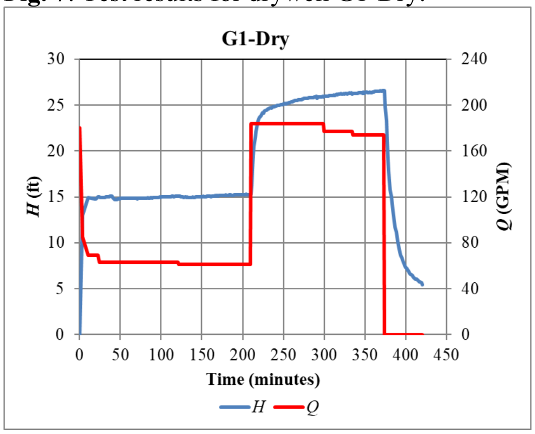


Table 5: Results of SSBP tests in G1-Dry.

Parameter	**Units**	**209 min.**	**370 min.**
Ponding head (H)	ft	15.3	26.5
Flow rate (Q)	gpm	61	174
Borehole radius (r_b)	in.	24	
Well casing radius (r_c)	in.	3	
Saturated sandpack length (L)	ft	15.3	26.5
Assumed sorptive number for med. sand (α^*)	1/ft	3.36	
Soil classification		SP	
Calculated hydraulic conductivity (K_b)	**ft/d**	**16**	**20**

3.3 Glendale Site 2

This section provides the infiltration testing result for the three wells installed at Glendale Site 2, including a 48-inch diameter drywell, an 8-inch diameter HSA well, and a 6-inch diameter sonic well. Based on the Sonic continuous core samples, which provided good recovery and relatively intact soil structure, the geology at the site consisted of the following:

- 0-13 ft: slightly silty to very silty fine SAND (SM and SM-SP)
- 13-18.5 ft: slightly silty, slightly gravelly, fine-coarse SAND with layers of very silty SAND (SP-SM and SM)
- 18.5-21 slightly sandy SILT (ML)
- 21-40 ft: trace silt to slightly silty, fine-coarse SAND with variable percentages of gravel, including cobbles from 30-40 ft (SP-SM and SP)
- 40-52 ft: fine-medium SAND with trace gravel and trace silt, silty sand layer at 49 ft (SP)
- 52-55 ft: silty fine-medium SAND (SM)

The wells were completed with sandpack and well screen from approximately 12-45 ft and were sealed with bentonite grout or bentonite chips above this interval.

3.3.1 *Well G2-Dry*

A single infiltration tests were conducted in G2-Dry on June 4, 2023. No well development was conducted before testing. Water was discharged into the 6-inch casing through 2-inch diameter drop pipe that was placed to a depth of 10 ft and the transducer was placed in the bottom of the 6-inch casing. As shown on Fig. 8, the test began with a target H = 13-14 ft and it was necessary to decrease the flow rate significantly during the first 100 minutes of the test to maintain that head elevation. After 238 minutes the flow rate was increased to the maximum flow allowed by the hydrant (169 gpm). The gaps in the H data are due to transducer issues during the test.

At 287 minutes the filter, meter and valve assembly at the wellhead was removed to reduce the head loss and this increased the maximum hydrant flow to 228 gpm. The flow rate was based on cumulative flow readings on the meter attached to the hydrant. Between 324 and 350 minutes the meter and valve assembly without the filter was replaced to see how much head loss was associated with the filter compared with the meter and valve. This resulted in a flow of 186 gpm, indicating that the filter caused 17 gpm of flow reduction and the meter and valve caused 42 gpm of flow reduction. The meter and valve assembly was removed again at 350 minutes and the water level rose gradually during the remainder of the test, indicating that steady-state conditions were not achieved by the end of the test at 459 minutes.

The results were analyzed using the methods described in Appendix C and summarized in Table 6. As indicated in the table, the initial portion of the test with H = 13.2 ft provided an estimated K_b value of 54 ft/d at 238 minutes. The results at 459 minutes and H = 19.9 ft provided an estimated K_b value of 41 ft/d. The reason for the decrease in capacity over time is likely due to mounding on the silty sand at a depth of 52 ft, 7 ft beneath the bottom of the well. This is confirmed by the relatively slow decline of the water level after the water was turned off and is further discussed in Section 3.3.4.

Fig. 8: Test results for drywell G2-Dry.

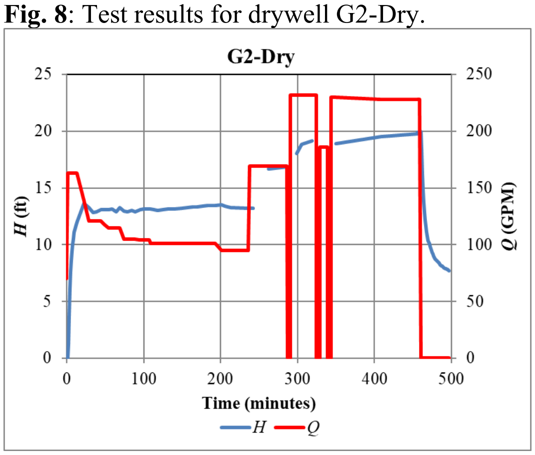


Table 6: Results of SSBP tests in G2-Dry.

Parameter	**Units**	**238 Min.**	**459 Min.**
Ponding head (H)	ft	13.2	19.9
Flow rate (Q)	gpm	169	228
Borehole radius (r_b)	in.	24	
Well casing radius (r_c)	in.	3	
Saturated sandpack length (L)	ft	13.2	19.9
Assumed sorptive number for med. sand (α*)	1/ft	3.36	
Soil classification		SP	
Calculated hydraulic conductivity (K_b)	**ft/d**	**54**	**41**

3.3.2 Well G2-HSA

The test in G2-HSA was conducted on June 4, 2023. Well development was conducted for 100 minutes before the test began. Approximately 200 gal of water was used during well development and the water was close to clear at the end of development.

The infiltration test conducted after well development is shown on Fig. 9. The test was started using a 20-200 gpm meter and it was necessary to turn the water off and on to avoid overflowing the top of the well. This continued for the first 40 minutes of the test and the flow rate shown on Fig. 9 represents the average flow during that time (calculated by dividing total flow by total duration). At 40 minutes the 20-200 gpm meter was replaced by a 5-50 gpm meter and this made it feasible to adjust the flow to maintain the water level at the top of the well casing. The test was close to steady state for the first 270 minutes and then the flow rate increased from 2.3 to 2.7 gpm in the last hour of the test while maintaining a constant head. The reason why the capacity increased towards the end of the test is uncertain, although it may be due to additional well development during the test.

The results were analyzed using the methods described in Appendix C. As indicated in Table 7, the test provided an estimated K_b value of 0.21 ft/d at 270 minute and 0.24 ft/d at 325 minutes. The ending K_b value of 0.24 ft/d is more than two orders of magnitude less than the K_b measured in G2-Dry and the results are not considered valid. Since this HSA well was completed in the same manner as G2-Sonic, the very poor performance of the HSA well is clearly due to clogging.

Fig. 9: Test results for test well B-HSA.

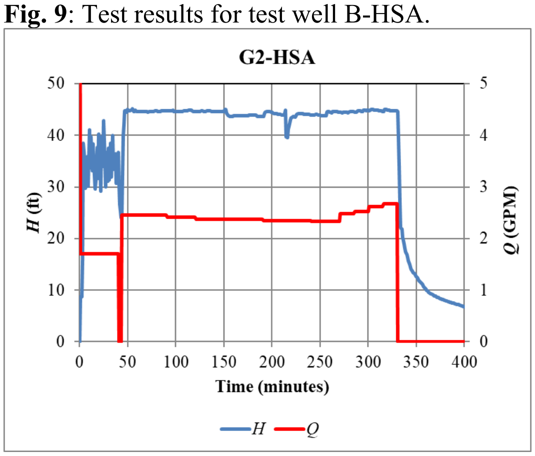


Table 7: Results of SSBP tests in test well B-HSA.

Parameter	**Units**	**270 min.**	**325 min.**
Ponding head (H)	ft	44.5	44.8
Flow rate (Q)	gpm	2.3	2.7
Borehole radius (r_b)	in.	4	
Well casing radius (r_c)	in.	1	
Saturated sandpack length (L)	ft	33	33
Assumed sorptive number for med. sand (α^*)	1/ft	3.36	
Soil classification		SP	
Calculated hydraulic conductivity (K_b)	**ft/d**	**0.21**	**0.24**

3.3.3 Well G2-Sonic

Two tests were conducted in G2-Sonic. Well development was conducted for approximately 25 minutes before the first test on June 3, 2023. Approximately 425 gallons of water was added during development and the water was clear when well development was terminated.

The first test, shown on Fig. 10(a) started with Q = 22 gpm and H = ~15 ft using a 20-200 gpm flow meter. At 69 minutes the 20-200 gpm meter was replaced with a 5-50 gpm flow meter and the flow was dropped to ~12 gpm so H would approximate H during the beginning of the test in G2-Dry (approximately 13 ft). At 200 minutes Q was increased to 32.5 gpm for the remainder of the test. As shown on Fig. 10(a), the test never reached steady-state and at the end of the test H = 19.6 ft.

The second test was conducted on June 5, 2023, and was designed to maintain the water level at the top of casing (H = ~46 ft). As shown on Fig. 10(b), the test reached steady-state within approximately 200 minutes with Q = 178 gpm. At ~350 minutes it was necessary to decrease the flow rain to maintain a constant head and by the end of the test Q was 166 gpm.

The results were analyzed using the methods described in Appendix C and summarized in Table 8. As indicated in the table, the first test provided an estimated K_b value of 13 ft/d at H = 14.5 ft and 200 minutes and an estimated K_b value of 14 ft/d at H = 19.6 ft and 380 minutes. The second test provided an estimated K_b value of 15 ft/d at H = 45.9 ft and 437 minutes. Although neither of the tests achieved steady state, the estimated K_b values were relatively similar. The estimated K_b values provided by G2-Sonic are significantly less than the estimated K_b values provided by G2-Dry. The reason for this difference is uncertain but could be explained by the groundwater mounding left over from the first test in G2-Dry, as discussed in Section 3.3.4.

Fig. 10: Test results for test well B-Sonic.

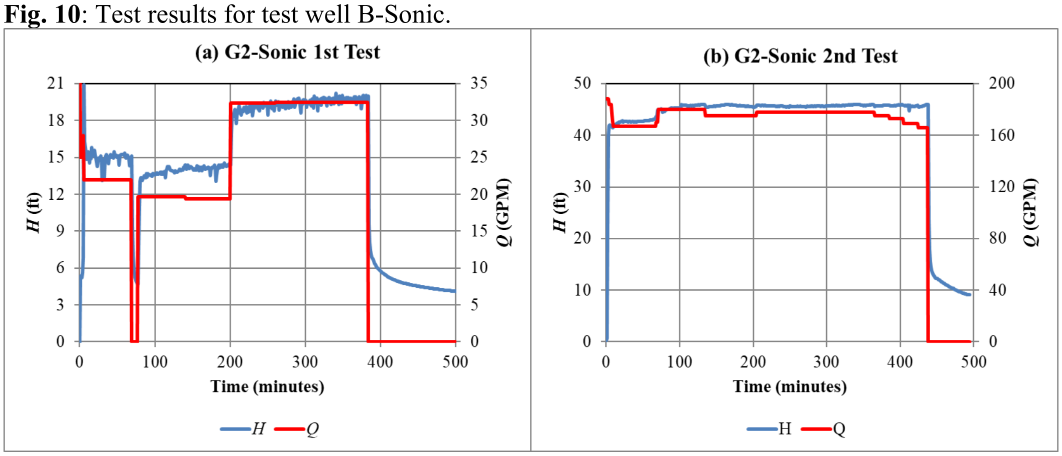


Table 8: Results of SSBP tests in test well B-Sonic.

Parameter	Units	1st Test 200 min.	1st Test 380 min.	2nd Test 350 min.
Ponding head (H)	ft	14.5	19.6	45.9
Flow rate (Q)	gpm	19.4	32.5	166
Borehole radius (r_b)	in.	4		
Well casing radius (r_c)	in.	1		
Saturated sandpack length (L)	ft	14.5	19.6	32
Assumed sorptive number for med. sand (α*)	1/ft	3.36		
Soil classification		SP		
Calculated hydraulic conductivity (K_b)	**ft/d**	**13**	**14**	**15**

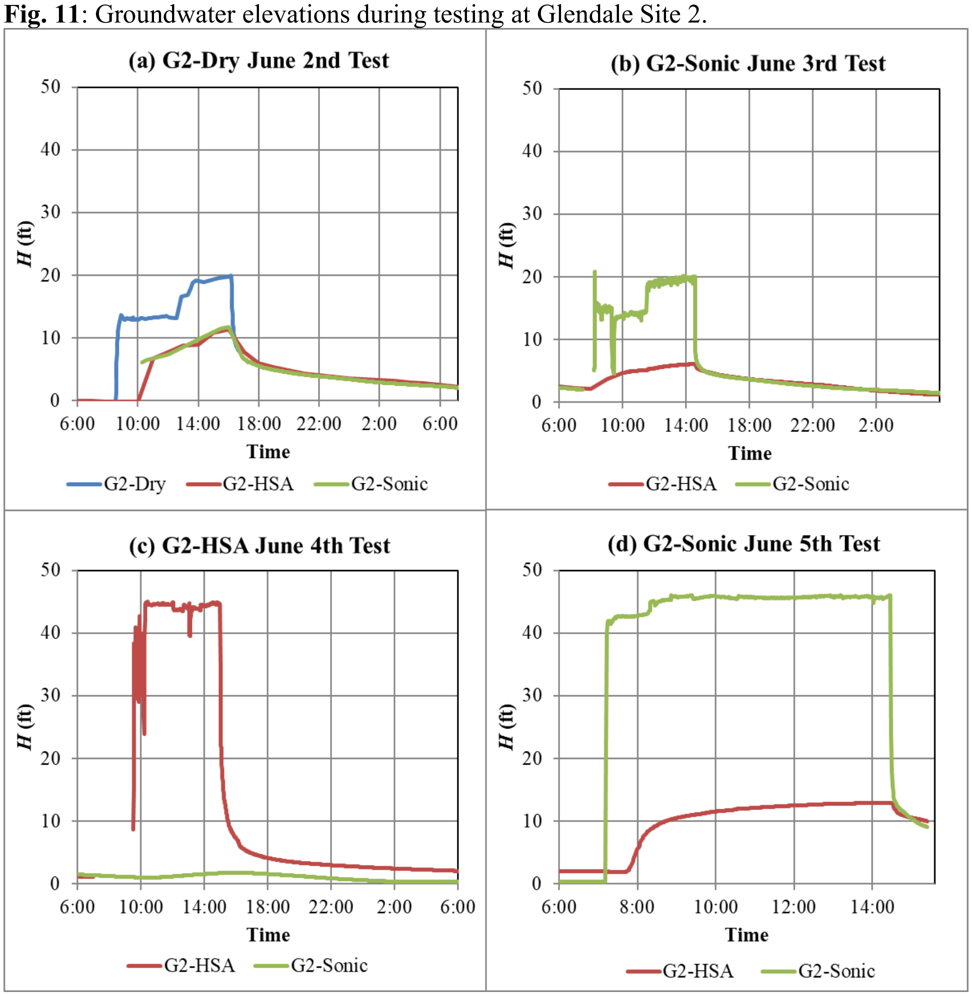
3.3.4 Groundwater Mounding

Water levels were monitored in the two test wells during the test in G2-Dry and in the other test well when tests were conducted in each of the test wells. It was not possible to monitor water levels in G2-Dry during tests in the test wells because G2-Dry was located in the street and that would have required shutting down a lane of traffic to open and close the drywell.

As shown in Fig. 11(a), water levels in G2-HSA and G2-Sonic rose approximately 11 ft during the test in G2-Dry on June 2nd. The water levels in all three well reached the same elevation shortly after the test ended and declined slowly overnight. As shown in Fig. 11(b), there was still ~ 2 ft of water in both test wells when the test in G2-Sonic began the next day. A similar pattern occurred following the first G2-Sonic test with ~5.9 ft of head rise in G2-HSA and a slow decline after the test was done. As shown in Fig. 11(c), the test in G2-HSA on June 4th resulted in a very minor head rise in G2-Sonic, reflecting the low flow rate in G2-HSA. As shown in Fig. 11(d), the high-head test in G2-Sonic on June 5th resulted in approximately 12 ft of head rise in G2-HSA.

The perching layer that caused the mounding shown in Fig.11 may be the silty sand layers observed at 49 ft and 52 ft. If the mound is perched on the layer at 52 ft, then the groundwater mound is another 7 ft higher than the depths shown on Fig. 11. This mound would have developed during the G2-Dry test and could be 18 ft thick, rather than the 11 ft shown on Fig. 11(a). Furthermore, the mound would still be 9 ft thick when the test in G2-Sonic started the next day. The presence of this mound could explain why the tests in G2-Sonic underestimated K_b in G2-Dry.

Fig. 11: Groundwater elevations during testing at Glendale Site 2.



4 Recommendations

This study provided the following lessons learned regarding installation and testing of drywells and small-diameter test wells.

- Confirming experience in the Ecology study, sonic wells do not appear to be clogged. However, the two side-by-side examples provided by this study suggest that sonic wells may underestimate the capacity of full-scale drywells.
- Confirming experience in the Ecology study, HSA wells are clogged and well development does not remove the clogging. HSA wells are not suitable for predicting the performance of full-scale drywells.
- Small-diameter test wells constructed with 2-inch diameter well screen may experience significant head loss in the screen and sandpack at high flow rates. This will provide K_b estimates that under-predict the capacity of full-scale drywells, which are typically constructed using 6-inch diameter well screen. This issue can be reduced by using 4-inch diameter well screen.
- Drop pipe that discharges below the water level is not necessary when the well is constructed using 6-inch diameter casing and screen. It is necessary when testing in 2-inch diameter casing (requires 1-inch diameter drop pipe) and recommended when testing in 4-inch diameter casing (suitable for 2-inch diameter drop pipe) to minimize air-entrainment.
- Because full-scale drywell capacity can be substantially greater than hydrant capacity, it is important to maximize the flow rate when testing small-diameter test wells in permeable soils. Hydrant flow rate can be maximized by using 2-inch diameter drop pipe rather than smaller diameter drop pipe and removing the filter, meter, and valve assembly at the well head. 2-inch diameter drop pipe requires at least 4-inch diameter well casing.
- The rate of falling head after the test can provide important information regarding the degree of perching and groundwater mounding. If the test well drains slowly relative to the steady-state test capacity, then more conservative correction factors should be used for estimating the capacity of full-scale drywells.

Although not an issue in this study, experience at other sites and discussions with Torrent Resources indicates that caving is a significant issue when drilling full-scale drywells using a solid-stem auger in clean sands and gravels. In many cases, it is not feasible to drill more than 10 ft into the permeable formation. Fortunately, although these soils may not stand open, they do provide high infiltration capacity. If such soils are encountered when drilling small-diameter test wells, it is recommended to complete the test well in the upper 10 ft of clean sand and gravel. This way, the test results will be useful for estimating the capacity of a full-scale drywell that has caving issues.

5 References

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Appendix A: Simulation of Large Diameter Boring Method in GS200.1 (LA County 2021)

A.1 Introduction

Numerical simulations were conducted to evaluate the accuracy of the large-diameter boring method in Los Angeles County *Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration* (LA County, 2021) and their ability to predict the performance of full-scale drywells. Both the falling-head large-diameter boring (FHLD) and the constant-head large-diameter boring (CHLD) methods were evaluated. These methods were compared with the uncased steady-state borehole permeameter (USSBP) and cased steady-state borehole permeameter (USSBP) methods described in Appendix C. Simulations were conducted for three soil types and compared with simulations of full-scale drywell capacity in the same soils.

Numerical simulations were conducted using SEEP/W, a two-dimensional, finite-element numerical model that can simulate axisymmetric flow in saturated and unsaturated porous media (GEOSLOPE International Ltd., Calgary, Alberta, Canada). Unsaturated flow simulation requires specifying soil hydraulic properties in the form of the unsaturated volumetric soil water content function $\theta(\psi)$ (soil water content as a function of soil matric suction) and the unsaturated hydraulic conductivity function $K(\psi)$ (hydraulic conductivity as a function of soil matric suction). These hydraulic property functions used for these simulations are described by Kindred (2022).

A.2 Model Domain, Test Configurations, and Boundary Conditions

The SEEP/W numerical flow domains for the test-well and drywell test configurations are shown in Fig. A-1. The simulated domain was 20 m deep with a radius of 6 m for the test well and 10 m for the drywell. For both the test well and the drywell, the well was 15 m deep, and the screen and filter pack were 9 m long. In other words, the upper 6 m of the wells were sealed, and the bottom 9 m were open to the formation. The test well had a borehole radius (r_b) = 0.25 m with a screen/casing radius (r_s) = 0.05 m. The drywell had r_b = 0.6 m with a r_s = 0.075 m.

The simulations used graded meshes of rectangular and triangular finite elements. Element size increased in steps from 0.01 m inside the well screen and casing to 0.3 m away from the borehole. Test simulations showed that larger flow domains and smaller element sizes had minimal impact on simulation results.

The test well simulations assumed that flow Q (m^3/d) was limited by the maximum capacity of a fire hydrant, which can vary from 750 - 1,200 m^3/d (138 – 220 gpm). A flux boundary condition of no more than 750 m^3/d was assumed for a single node near the bottom of the well screen and water was allowed to exit the well casing either at the top of the screen (uncased scenarios) or at the ground surface (cased scenario) using a seepage face boundary condition[1]. Flow did not exit the well casing until the ponding depth (H) rose to either the top of the screen (uncased scenarios) or at the ground surface (cased scenario). The total flow into the formation was tracked by subtracting the flow out of the well casing from the flow into the node at the bottom of the screen. A seepage face boundary condition was also applied at the outer edge of the axisymmetric domain (the right-hand side) and a unit hydraulic gradient[2] boundary condition was applied at the bottom of the domain.

Drywell simulations were designed to estimate the flow when the water level was at the top of the screen (H = 9 m) for the uncased scenario and when the water level was 1.0 m below the top of the casing (H = 14 m) for the cased scenario. These simulations utilized a fixed-head boundary condition at the inside of the well screen (the left-hand side) to estimate simulated Q during a storm event.

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[1] A seepage face boundary condition is assumed to be a flux-boundary (set to zero in these simulations) until water pressure exceeds zero, at which point it becomes a fixed head boundary conditions set equal to the node elevation and water is allowed to exit the model.

[2] A unit hydraulic gradient boundary condition assumes pore-water pressure is constant with depth. This means the hydraulic gradient = elevation gradient, which is always 1, and flux through the boundary is due entirely to gravity flow. In this scenario, flux equals unsaturated hydraulic conductivity (which is a function of the moisture content), and the boundary is simulated as a flux boundary condition that varies with moisture content.

The numerical simulations calculated water flow rate or discharge, Q (m^3/d), into the formation, cumulative Q, and H in the well. The infiltration rate was calculated by dividing Q by the saturated area of the borehole (*AREA*), including both the bottom and sidewalls where:

$$AREA = \pi r_b^2 + 2\pi r_b H \qquad Eq. A.1$$

A.3 Representative Soil Types

Kindred (2022) defined ten "representative" soils for calibration of the SSBP fitting parameters, including five glacially over-consolidated soils and five normally consolidated soils (typical of recessional outwash or alluvium). These soils can also be used for numerical simulations. The three soils used for this analysis include silty fine sand, medium sand, and sandy gravel, with saturated hydraulic conductivity (K_s) of 0.25, 10, and 30 m/d, respectively. The material properties for the soils used in the simulations are summarized in Table A-1. In addition, Kindred (2022) defined material properties for bentonite, well casing, and filter pack for use in numerical simulation of the materials inside the borehole. The bentonite was defined using parameters suitable for clay, the volume inside the well casing was defined as 100% porosity and $K_s = 1x10^6$ m/d, and the filter pack was defined as very permeable sand with porosity of 40% and $K_s = 1x10^4$ m/d. The well casing and sandpack materials provide negligible resistance to flow, even at very high flow rates. Although this eliminated the effects of the screen and sandpack from the numerical simulations, it is possible that head loss through the screen and sandpack may affect capacity in field tests.

Table A-1: Properties of native soils used in the numerical simulations. D_{60} and D_{10} are grain diameters corresponding, respectively, to 60% passing and 10% passing on the grain-size distribution curve, and *USCS* is Unified Soil Classification System. *Background soil matric suction* is based on the assumed *background soil water content* shown in the table.

Parameter	**Soil Type**		
	Silty Fine Sand	**Medium Sand**	**Sandy Gravel**
D_{60} (mm)	0.15	1.0	8.0
D_{10} (mm)	0.04	0.18	0.4
Silt Content (wt. %)	25%	5%	3%
USCS Soil Type	SM	SP	GW
Porosity, θ_S (vol. %)	40	40	40
Liquid Limit (%)	10	0	0
Saturated Hydraulic Conductivity, K_s (m/d)	0.25	10	30
Residual Soil Water Content, θ_r (vol. %)	4.8	2.2	1.3
Background Soil Water Content θ_b (%)	9.8	7.2	6.3
Background Soil Matric Suction, ψ_i (m)	1.39	0.24	0.05
van Genuchten Fitting Parameter α' (m^{-1})	1.28	7.69	40
van Genuchten Fitting Parameter n (-)	4.3	4.3	3.9

A.4 FHLD Test Analysis

The analysis method provided by GS200.1 (LA County, 2021) for the FHLD method is unclear and includes the following language: "The field infiltration rate must account for non-vertical flow through the sides of the boring in addition to the bottom of the boring." Since the test procedures calls for documenting both the drop in water and the volume during each refill cycle, this analysis assumes that the field infiltration rate (I) can be calculated using the following equation:

$$I = \frac{V}{AREA\ x\ T} \qquad \text{Eq. A. 2}$$

V = volume of water added during the refill cycle, *AREA* is calculated using Eq. A.1 and the average *H* during the refill cycle, and *T* = duration of the refill cycle (both fill time and falling head time interval). This method assumes that the well is refilled to approximately the same *H* each refill cycle.

A.5 Test Scenarios

The test scenarios were designed to mimic the FHLD and CHLD methods in GS200.1 (LA County, 2021), the SSBP test methods developed by Kindred (2022), and actual capacity of a drywell. Each scenario was performed for all three soil types. The scenarios are described below:

FHLD test:

Step 1: Pre-soak for 1 hr at *H* = 9 m or *Q* = 750 m^3/d, whichever is limiting.
Step 2: Falling head for 30 minutes or when the well is empty, whichever occurs first.
Step 3: Refill well for 6 minutes at *H* = 9 m or *Q* = 750 m^3/d, whichever is limiting.
Steps 4-6: Repeat Steps 2 and 3 three more times, for a total test duration of up to 3.4 hours.
Analysis: Analyze results in accordance with the method described in the previous section to obtain an infiltration rate. Use the calculated infiltration rate to estimate the capacity of a full-scale drywell with *H* at or above the top of the well screen.

CHLD test:

Step 1: Pre-soak for 1 hr at a max. *H* = 9 m or max. *Q* = 750 m^3/d, whichever is limiting.
Step 2: Falling head for 30 minutes.
Step 3: Attempt to maintain steady-state conditions in the well for 3 hours at *H* = 9 m or *Q* = 750 m^3/d, whichever is limiting. Total test duration of 4.5 hours.
Analysis: Analyze results in accordance with GS200.1 (LA County, 2021) to obtain an infiltration rate. Use the calculated infiltration rate to estimate the capacity of a full-scale drywell with *H* at or above the top of the well screen.

USSBP test:

Test Procedure: Attempt to maintain steady-state conditions in the well for 6 hours at *H* = 9 m or *Q* = 750 m^3/d, whichever is limiting.
Analysis: Analyze results in accordance with the USSBP method Appendix C to obtain a "measured" K_s estimate. Use the "measured" K_s to estimate the capacity of a full-scale drywell with *H* below the top of the well screen.

CSSBP test:

Test Procedure: The CSSBP test was only conducted for silty fine sand because *H* did not rise above the screen for the other two soils at the maximum hydrant capacity of 750 m3/d. This method attempts to maintain steady-state conditions in the well for 6 hours at *H* = 14 m or *Q* = 750 m^3/d, whichever is limiting.
Analysis: Analyze results in accordance with the CSSBP method in Appendix C to obtain a "measured" K_s estimate. Use the "measured" K_s to estimate the capacity of a full-scale drywell with *H* at or above the top of the well screen.

Drywell simulation:

Test Procedure: Maintain constant-head in the well for 6 hours with both *H* = 9 m (uncased scenario) and *H* = 14 m (cased scenario). These scenario was intended to simulate a large storm with peak runoff for 6 hours. Inflow to the drywell is not limited by hydrant capacity.
Analysis: Analyze results in accordance with Appendix C to obtain a "measured" K_s estimate for comparison with the K_s specified in the numerical model.

A.6 Results

Numerical simulations were conducted to evaluate the accuracy of the FHLD and CHLD methods in GS200.1, (LA County, 2021) and the SSBP methods described in Appendix C. Simulations were conducted for three soil types and compared with simulations of full-scale drywell capacity in the same soils.

FHLD Results:

The FHLD results are summarized in Table A-2 and illustrated in Figs. A-2 and A-3. Fig. A-2 shows the zero-matrix suction (i.e., water pressure) line and the water content contours for the three soils and the end of the third refill cycle. Fig. A-3 shows H during the entire test (pre-soak period and three refills) for each soil type. As shown in the figures, the test in silty fine sand was limited by $H \leq 9$ m, while the test in medium sand and sandy gravel were limited by $Q \leq 750$ m^3/d. As a results, H never rose above ~6 m in medium sand and ~3 m in sandy gravel.

Table A-2: Numerical simulation results for FHLD test in three soil types. V = volume of water per refill cycle, *AREA* = average saturated area during the refill cycle, T = duration of each refill cycle, I = infiltration rate. *Cumulative Volume* is the total amount of water used during the test.

Soil	Presoak Time Interval (min)	Ave. V (m^3)	Ave. H (m)	AREA (m^2)	T *(min)*	I (m/d)	Cumulative Volume (m^3)
Silty fine sand	30	0.42	6.7	10.7	36	1.6	7.4
Medium sand	25	3.1	3.1	5.0	31	29	41
Sandy gravel	4	3.1	1.6	2.7	18	92	41

CHLD Results:

The CHLD results are summarized in Table A-3 and illustrated in Figs. A-4 and A-5. Fig. A-4 shows the zero-matrix suction (i.e., water pressure) line and the water content contours for the three soils and the end of the constant-head test. Fig. A-5 shows H during the entire test (pre-soak period and 3 hr of constant head) for each soil type. Similar to the falling-head tests, the test in silty fine sand was limited by $H \leq 9$ m, while the test in medium sand and sandy gravel were limited by $Q \leq 750$ m^3/d.

Comparison of Table A-2 and A-3 illustrate that the constant-head tests used considerably more water than the falling-head tests, ranging from an increases of 68% for the silty fine sand to 200% for the medium sand and sandy gravel. This increase indicates that the constant-head tests saturate a much large volume of soil and are more likely to be affected by mounding on perching layers. In addition, the constant-head tests provide a significantly higher estimate of I than the falling-head tests, demonstrating that the results from these two types of tests should not be considered comparable.

Table A-3: Numerical simulation results for CHLD test in three soil types. Final H = hydraulic head or ponding depth at the end of the test, final Q = flow at the end of the test, *AREA* = saturated area at the end of the test, I = calculated infiltration rate. *Cumulative Volume* is the total amount of water used during the test.

Soil	Presoak Time Interval (min)	Final H (m)	Final Q (m^3/d)	AREA (m^2)	I (m/d)	Cumulative Volume (m^3)
Silty fine sand	30	9.1	50	14.4	3.5	12.5
Medium sand	25	6.9	750	11.0	68	125
Sandy gravel	4	3.4	750	5.6	134	125

USSBP and CSSBP Test Results:

The USSBP and CSSBP results for the test well are summarized in Table A-4 and illustrated in Figs. A-6 and A-7. Fig. A-6 shows the zero-matrix suction (i.e., water pressure) line and the water content contours at the end of each test for the three USSBP tests and the CSSBP test in the silty fine sand. It was not possible to simulate CSSBP tests in the medium sand and the sandy gravel because the flow was already at the maximum hydrant capacity of 750 m^3/d for the USSBP tests. The higher head for the CSSBP test in the silty fine sand results in a larger zone of saturation than the USSBP test in the same soil.

Fig. A-7 shows H during the entire 6-hr USSBP tests for each soil type. The test in silty fine sand has achieved the target of $H = 9$ m and the flow needs to decrease to maintain that H values (as shown on Fig. A-8). The tests in medium sand and sandy gravel have not reached the target $H = 9$ m but have obtained a steady state H at the maximum flow of 750 m^3/d. Fig. A-8 shows Q for the two tests in silty fine sand during the 6-hr test. As shown in the figure, the flow is still decreasing at the end of the tests in silty fine sand and have not reached steady-state by the end of 6 hr. As shown in Kindred (2022), it is typical to not achieve steady-state conditions in fine-grained soils by 6 hr.

Table A-4 provides H and Q at the end of each test, cumulative water volume used for each test, and "measured" K_s estimated for each test using the USSBP and CSSBP methods provided in Appendix C. Table A-4 also provides the specified K_s used in the numerical simulations. The "measured" K_s estimates for silty fine sand overestimate the specified K_s by 40-52%. One reason for this discrepancy is that the tests did not achieve steady-state conditions by the end of the test and calibration of fitting parameters was based on 24-hr tests. Steady-state conditions were achieved for the medium sand and sandy gravel by the end of the tests and the "measured" K_s estimates were within 3-9% of specified K_s.

Table A-4: Numerical simulation results for SSBP tests in three soil types. Final H = hydraulic head or ponding depth at the end of the test, final Q = flow at the end of the test, *SSBP* K_s is hydraulic conductivity calculated from the numerical results and the USSBP and CSSBP methods provided in Appendix C. *Cumulative Volume* is the total amount of water used during the test.

Soil	Max. H (m)	Final Q (m^3/d)	Cumulative Volume (m^3)	SSBP K_s (m/d)	Specified K_s (m/d)
Silty fine sand (USSBP)	9.1	48	15.6	0.38	0.25
Silty fine sand (CSSBP)	15.0	90	41.0	0.35	0.25
Medium sand	6.9	750	187	9.66	10
Sandy gravel	3.4	750	187	27.4	30

Simulated Drywell Test Results:

The USSBP and CSSBP results for the drywell are summarized in Tables A-5 ($H = 9$ m) and A-6 ($H = 14$ m) and illustrated in Figs. A-9 and A-10. Fig. A-9 shows the zero-matrix suction (i.e., water pressure) line and the water content contours at the end of each drywell simulation for $H = 9$ m and $H = 14$ m. This figure illustrates how much further the saturated zone can spread from a full-scale drywell compared with the infiltration tests illustrated in Figs. A-2, A-4, and A-6. Fig. A-10 shows Q for the entire 6-hr drywell simulations at $H = 9$ m and $H = 14$ m and the three soil types. As shown in the figure, the flow is still decreasing at the end of the tests in silty fine sand and have not reached steady-state by the end of 6 hr. This is the same pattern as shown in Fig. A-8 for the SSBP tests in silty fine sand. In contract for medium sand and sandy gravel have achieved steady state by the end of the s hr simulations.

Tables A-5 and A-6 provide H and Q at the end of each test, cumulative water volume used for each test, and "measured" K_s estimated for each test using the USSBP and CSSBP methods provided in Appendix C. The tables also provide the specified K_s used in the numerical simulations. The "measured" K_s estimates for silty fine sand overestimate the specified K_s by 44-60%, similar to the results for the test well, and reflect that steady-state conditions have not been achieved at the end of 6 hr. Steady-state conditions were achieved for the medium sand and sandy gravel by the end of the tests and the "measured" K_s estimates were within 6-12% of specified K_s.

Table A-5: Numerical simulation results for full-scale drywell with H = 9 m (uncased scenario) in three soil types.

Soil	Max. H (m)	Q (m^3/d)	*USSBP* K_s (m/d)	Specified K_s (m/d)	Cumulative Volume (m^3)
Silty fine sand	9	75.3	0.40	0.25	31
Medium sand	9	1,689	9.34	10	474
Sandy gravel	9	4,933	27.5	30	1,288

Table A-6: Numerical simulation results for full-scale drywell with H = 14 m (cased scenario) in three soil types.

Soil	Max. H (m)	Q (m^3/d)	*CSSBP* K_s (m/d)	Specified K_s (m/d)	Cumulative Volume (m^3)
Silty fine sand	14	128	0.36	0.25	48
Medium sand	14	3,082	9.05	10	861
Sandy gravel	14	8,969	26.5	30	2,348

Ability of Test Methods to Predict Drywell Capacity:

The predicted drywell capacity based on test results provided by the GS200.1 and the SSBP methods are summarized in Table A-7 and compared with the simulated drywell capacities for H = 9 m and H = 14 m. The same comparisons are provided in Figure A-10. One limitation of the GS200.1 methods is that they do not account for $H > L$, so the predicted capacity is the same for H = 9 m and H = 14 m. The SSBP results shown on Fig. A-10 are based on the USSBP method for H = 9 m and the CSSBP method for H = 14 m.

As shown in Table A-7 and Fig. A-10, the FHLD method underpredicts drywell capacity in all cases, with error ranging from -27% to -40% for H = 9 m. The errors are much greater when H = 14 m since this method does not account for drywell capacity where $H > L$.

The CHLD method underpredicts drywell capacity in some cases and overpredicts drywell capacity in others, with error ranging from -5% to +64% when H = 9 m. In one scenario (silty fine sand with H = 14 m) the CHLD method is relatively accurate, although this is by coincidence and shouldn't be viewed as a typical result. This method does not account for the full dynamics of unsaturated flow from a well, including capillarity and effects of pressure head.

As shown in Table A-7, the USSBP and CSSBP methods provide relatively accurate estimates of drywell capacity, with errors ranging from -4% to +3%. These methods did not accurately measure specified K_s in silty fine sand (because steady-state conditions were not achieved during the test) but they do a good job of predicting drywell capacity for an intense storm of similar length as the test duration. For this analysis, this is because the drywell also doesn't attain steady-state conditions in six hours for silty fine sand. In theory, drywell capacity may be less than predicted during a particularly long intense storm, but maximum runoff rates are rarely sustained for more than 6 hr. These methods did provide relatively accurate estimates of specified K_s in coarse-grained soils and even better estimates of drywell capacity.

Table A-7: Predicted drywell capacity based on different test methods and simulated drywell capacity for three soil types.

Soil	Predicted Drywell Capacity (m^3/d)				Simulated Drywell (m^3/d)	
	Falling-Head GS200.1 (FHLD)	Constant-Head GS200.1 (CHLD)	*USSBP* (H = 9 m)	*CSSBP* (H = 14 m)	Drywell (H = 9 m)	Drywell (H = 14 m)
Silty fine sand	54.5	123	72	127	75	128
Medium sand	1,020	2,397	1,747	3,119	1,689	3,082
Sandy gravel	3,219	4,683	4,915	8,801	4,933	8,969

A.7 Discussion:

This analysis compared the ability of the large-diameter boring methods in GS200.1 and the SSBP methods (Appendix C) to predict full-scale drywell capacity. This analysis demonstrated that neither the FHLD or CHLD methods are very accurate. For the soils and scenarios tested in this analysis, errors ranged from -40% to +64%, assuming that $H \leq L$. Errors were considerably larger when $H \geq L$, which is a significant issue since most drywells are sealed to a depth of 10 ft or more and maximum capacity will be achieved when H approaches the ground surface. The fundamental issue with the GS200.1 methods is that they provide an estimate of infiltration rate, which is a function of both the soil and the geometry of the test facility, and do not account for the full dynamics of unsaturated flow from a well.

GS200.1 specifies that the large-diameter boring methods should be conducted with H approximately equal to maximum H in the drywell. However, typical hydrant capacity is not sufficient to achieve high H in permeable soils commonly encountered in the Los Angeles basin. Therefore, this element of the methods is difficult to achieve.

The SSBP methods described in Appendix C did provide accurate estimates of drywell capacity, with errors ranging from -4% to +3% for both $H \leq L$ and $H \geq L$. These methods will have the same challenge with hydrant capacity as the GS200.1 methods. However, these methods provide an estimate of K_s, which is truly a soil parameter, and the methods account for the full dynamics of unsaturated flow from a well. Therefore, as long as K_s for the native soils is the same for the entire sandpack interval, these methods will accurately predict drywell capacity for any values of H and L.

This analysis was conducted using numerical simulations that assumed isotropic and homogeneous K_s throughout the entire simulated domain. Furthermore, the SSBP method assumes isotropic and homogeneous K_s. In the real world, soils are layered and variable and SSBP errors will be significantly greater than demonstrated in this analysis.

A.8 References

Kindred JS (2022) Final Report: Near-Term Action (NTA) 2018-0827: Flexible Infiltration Test Methods for Evaluating Infiltration Feasibility, Prepared for: City of Tacoma, October 10, 2022, available at: https://kindredhydro.com/infiltration-downloads-1

LA County (2021) Guidelines for Geotechnical investigation and reporting low impact development stormwater infiltration, Los Angeles County Public Works Administration Manual GS200.1, June 30, 2021, available at: http://dpw.lacounty.gov/gmed/permits/docs/policies/GS200.1.pdf

Fig. A-1: SEEP/W axisymmetric model domains and boundary conditions for the test well and drywell configurations.

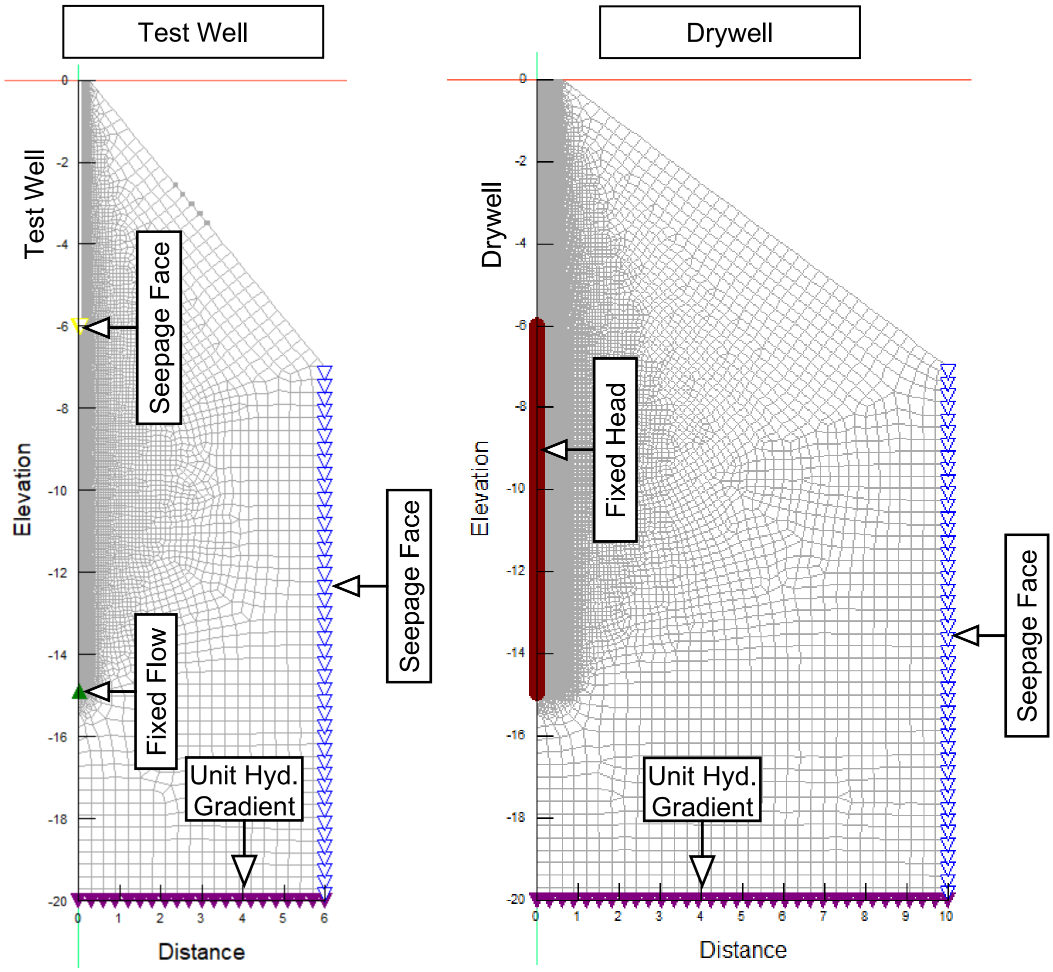


Fig. A-2: Zero matric suction and water content contours for the FHLD simulations at the end of the third refill cycle for three soil types. Test were conducted with $H \leq 9$ m or $Q \leq 750$ m^3/d, whichever was limiting. The dashed blue line indicates zero matrix suction (i.e., water pressure).

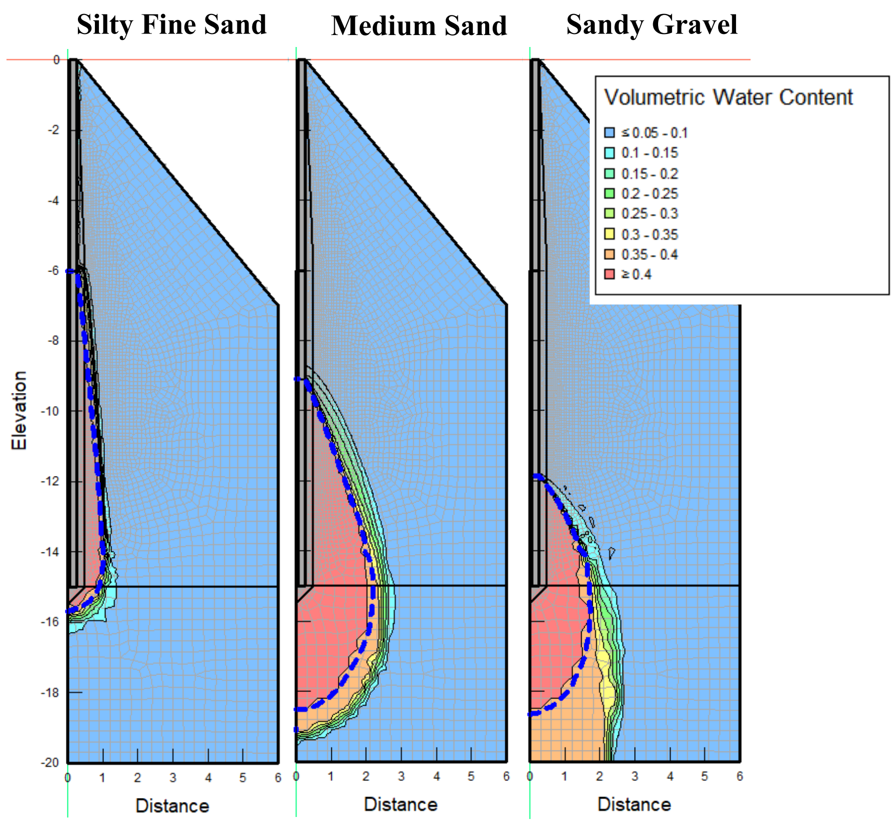


Fig. A-3: Pressure head for the FHLD simulations for three soil types. Test were conducted with $H \leq 9$ m or $Q \leq 750$ m^3/d, whichever was limiting.

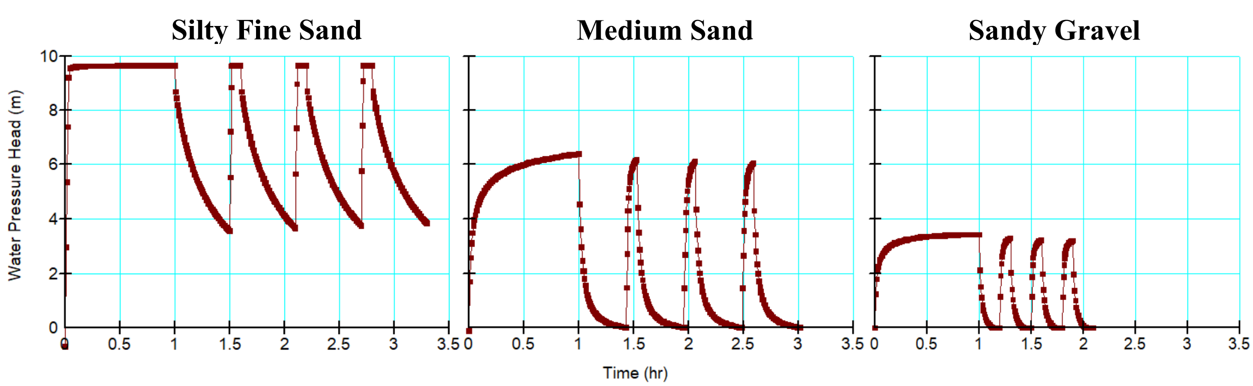


Fig. A-4: Zero matric suction and water content contours for the CHLD simulations at the end of the test for three soil types. Test were conducted with $H \leq 9$ m or $Q \leq 750$ m^3/d, whichever was limiting. The dashed blue line indicates zero matrix suction (i.e., water pressure).

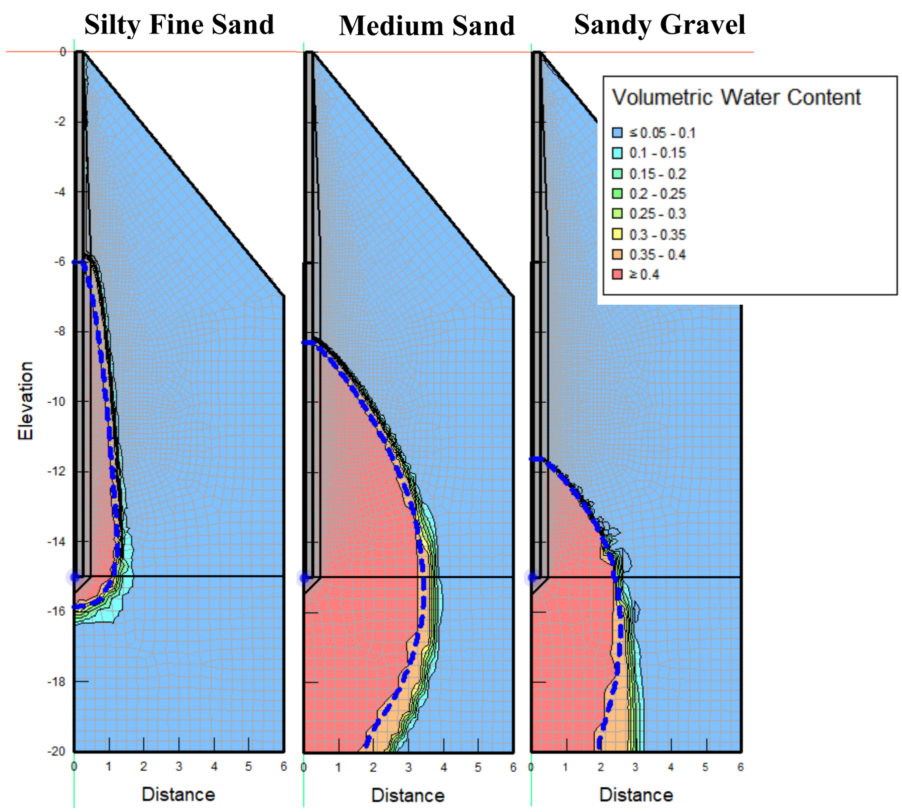


Fig. A-5: Pressure head for the CHLD simulations for three soil types. Test were conducted with $H \leq 9$ m or $Q \leq$ 750 m^3/d, whichever was limiting.

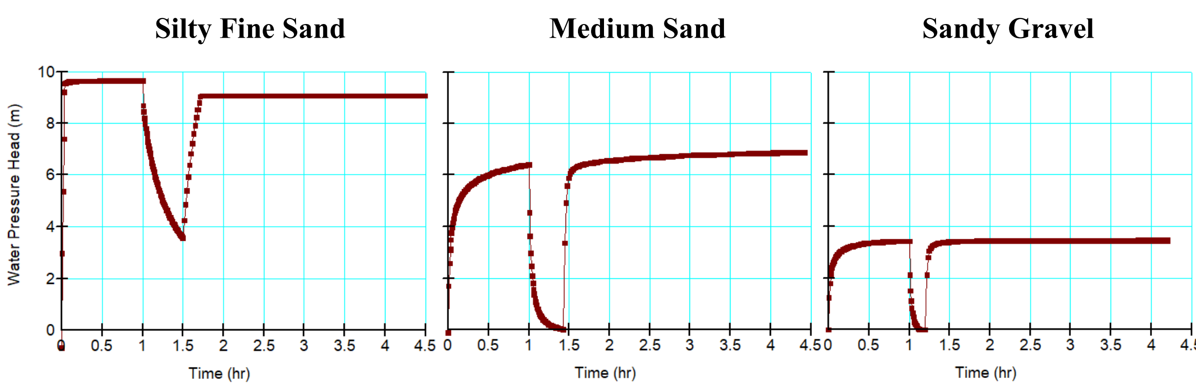


Fig. A-6: Zero matric suction and water content contours for the USSBP and CSSBP test simulations at the end of the test for three soil types. Test were conducted with $H \leq 9$ m (USSBP) or $H \leq 15$ m (CSSBP) and $Q \leq 750$ m^3/d, whichever was limiting. The dashed blue line indicates zero matrix suction (i.e., water pressure).

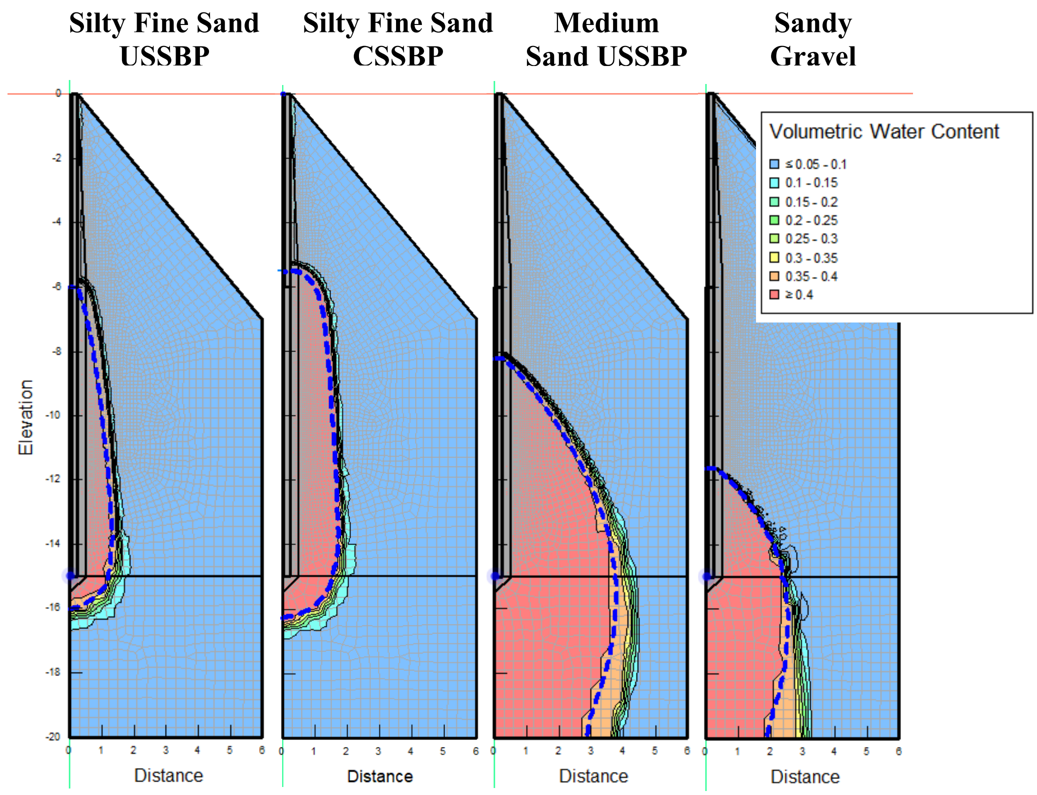


Fig. A-7: Pressure head (H) and flow (Q) for the USSBP simulations for three soil types. Test were conducted with $H \leq 9$ m and $Q \leq 750$ m^3/d, whichever was limiting.

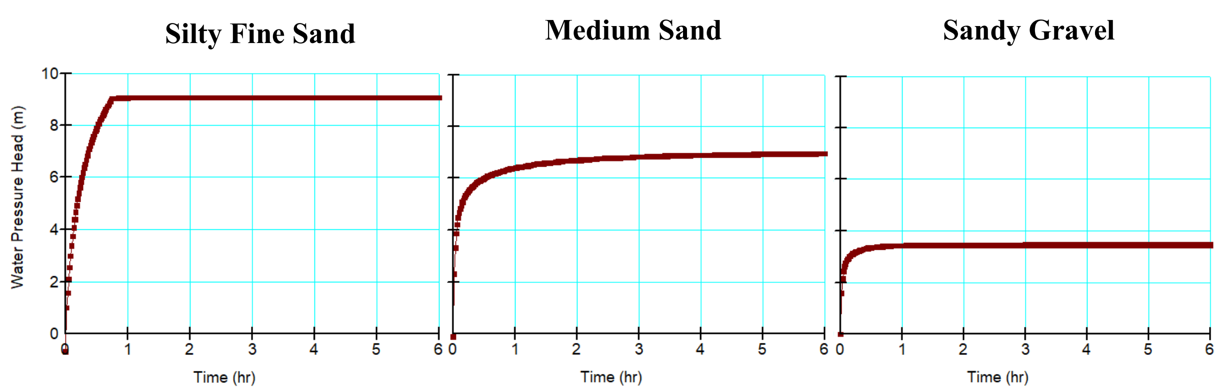


Fig. A-8: Water rate (Q) for the USSBP and CSSBP simulations in silty fine sand. The USSBP test was conducted with $H \leq 9$ m and the CSSBP test was conducted with $H \leq 15$ m.

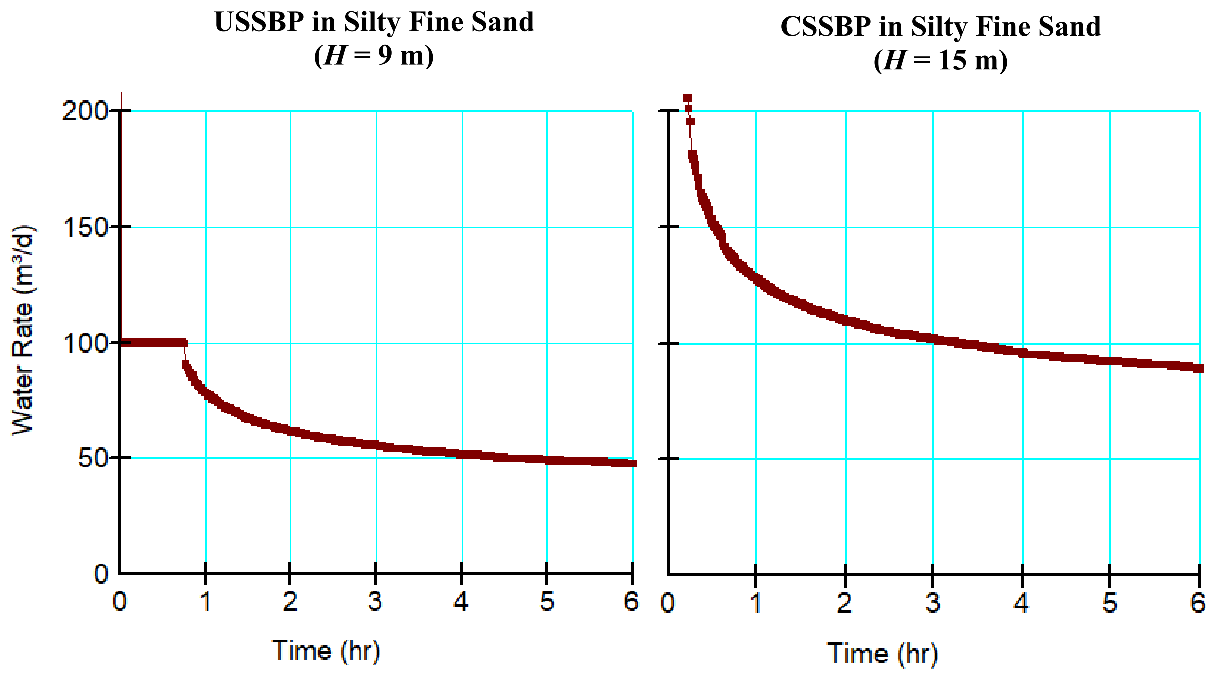


Fig. A-9: Zero matric suction and water content contours for the drywell at the end of each 6-hr simulation. Results provided for $H = 9$ m and $H = 14$ m for three soil types. The dashed blue line indicates zero matrix suction (i.e., water pressure).

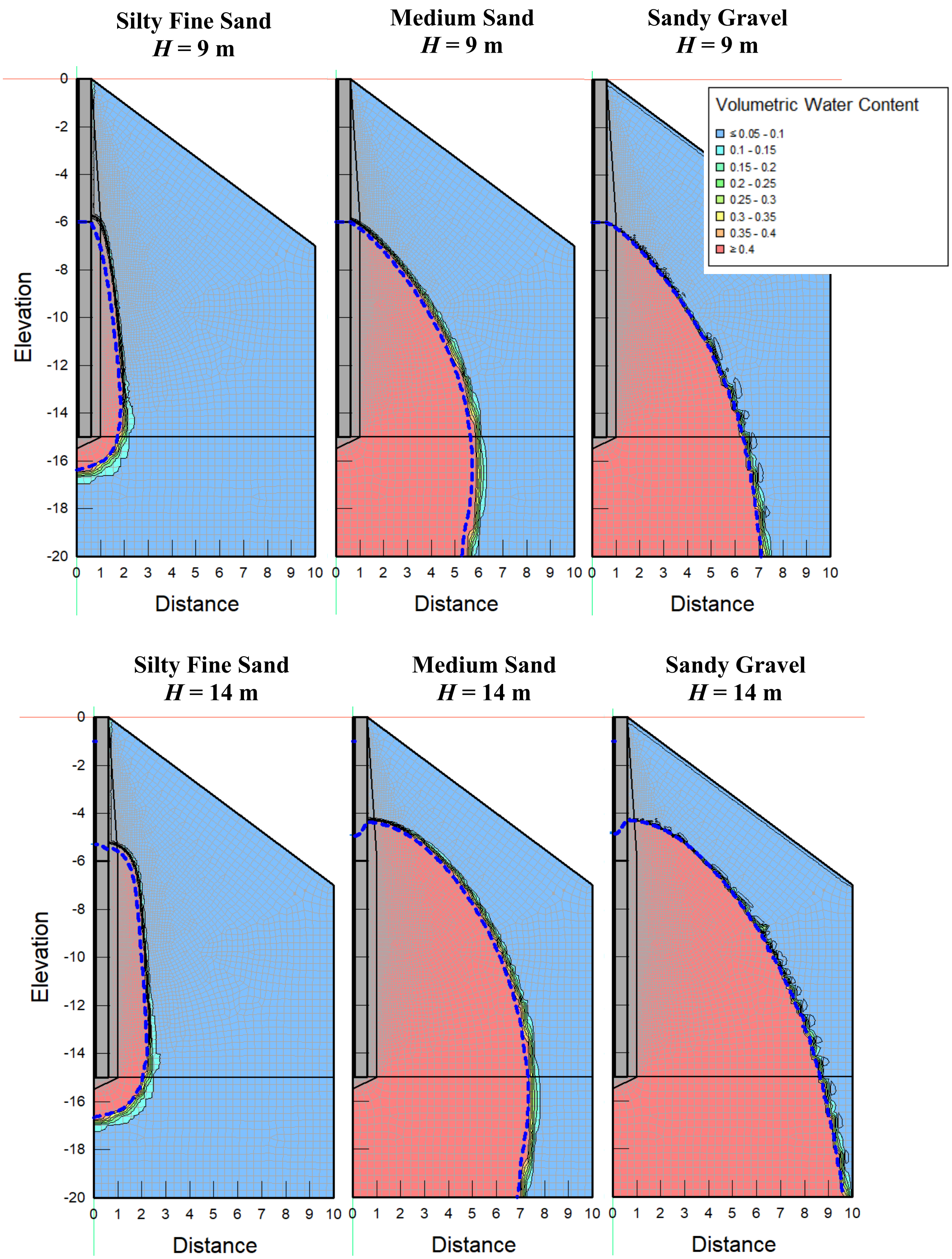


Fig. A-10: Flow (Q) for the drywell simulations for $H = 9$ m and $H = 14$ m for three soil types.

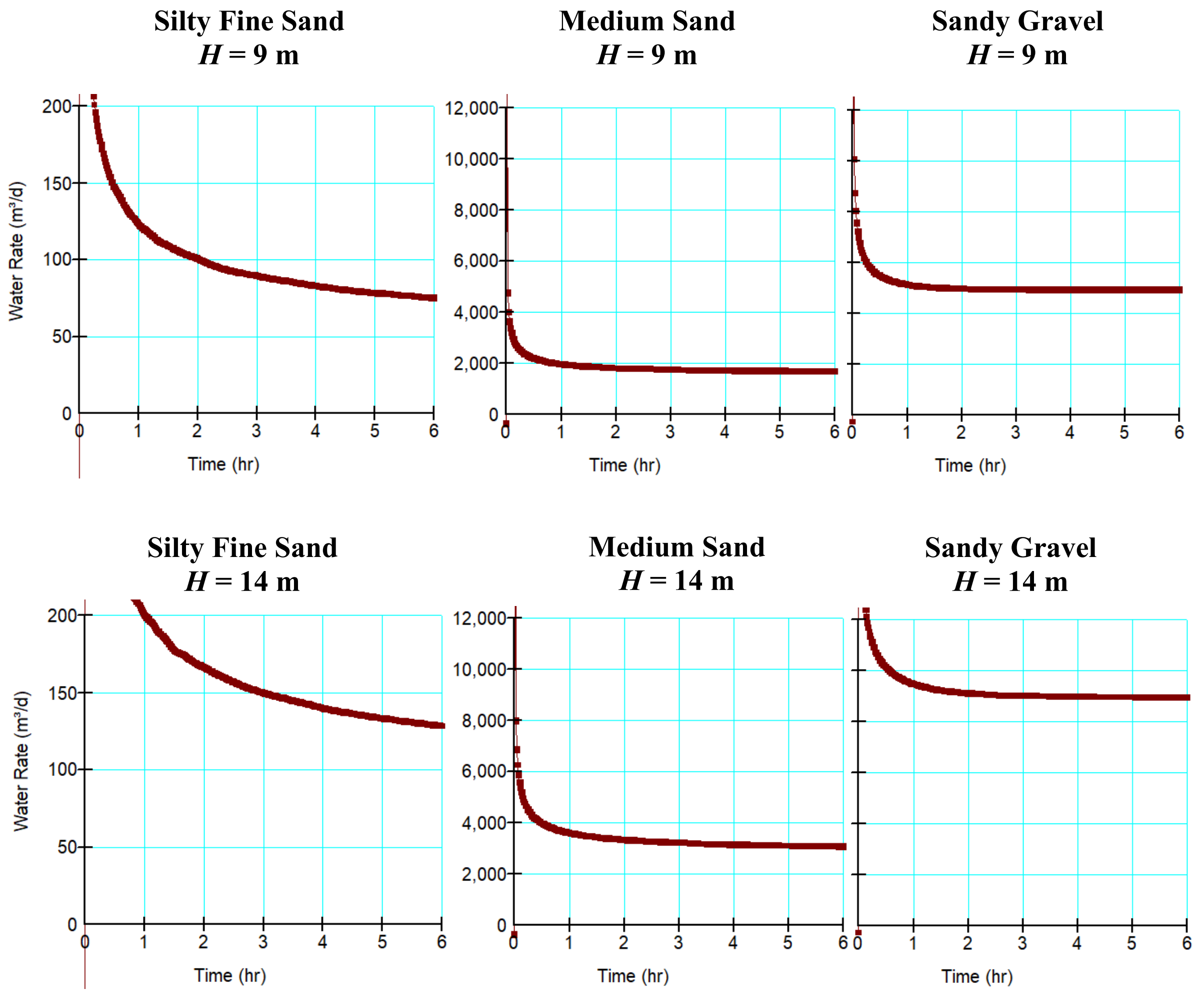


Fig. A-8: Zero matric suction and water content contours for the USSBP and CSSBP test simulations at the end of the test for three soil types. Test were conducted with $H \leq 9$ m (USSBP) or $H \leq 15$ m (CSSBP) and $Q \leq 750$ m^3/d, whichever was limiting. The dashed blue line indicates zero matrix suction (i.e., water pressure).

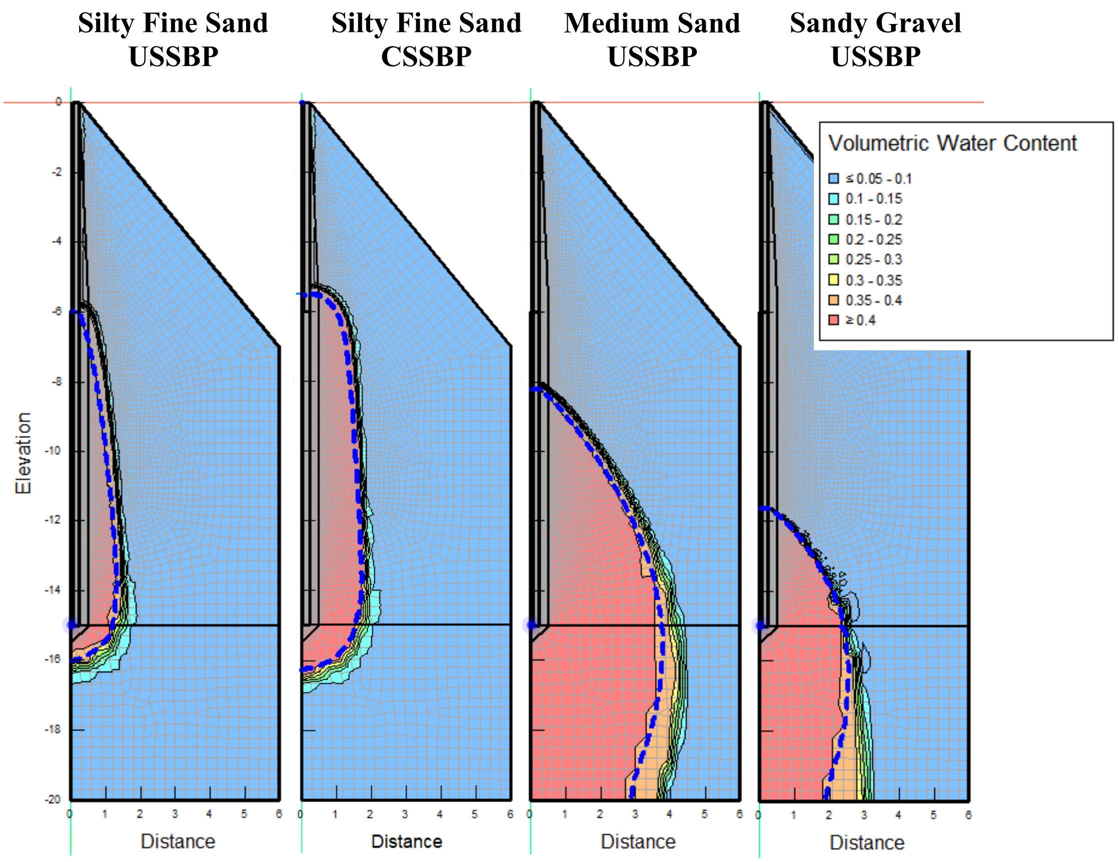


Fig. A-9: Pressure head for the USSBP and CSSBP simulations for three soil types. Test were conducted with $H \leq 9$ m (USSBP) or $H \leq 15$ m (CSSBP) and $Q \leq 750$ m^3/d, whichever was limiting.

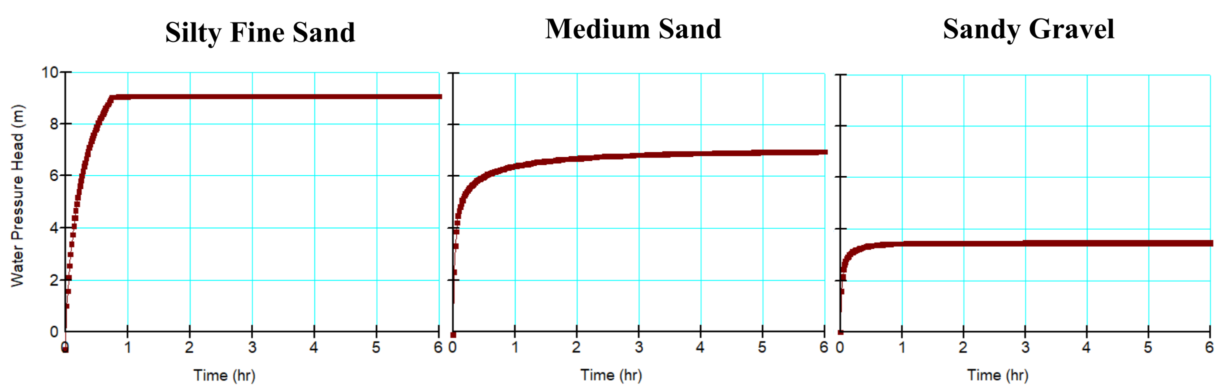
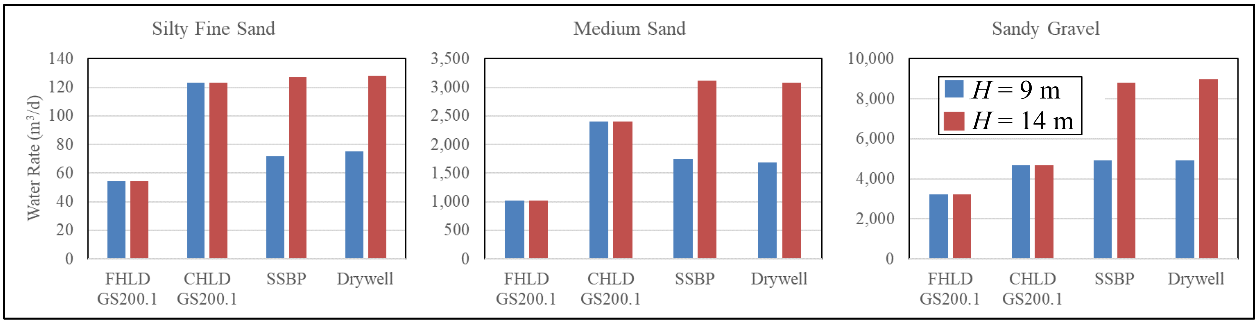


Fig. A-10: Estimates of drywell capacity provided by the GS200.1 falling-head large-diameter boring (FHLD), GS200.1 constant-head large-diameter boring (CHLD), and steady-state borehole permeameter (SSBP) methods, compared with numerical simulation of drywell capacity. Comparisons are shown for three soil types and with $H = 9$ m or $H = 15$ m.



Appendix B: A Literature Review of Infiltration Testing Methods in the United States

B.1 Introduction

Urban development results in the replacement of native vegetation and soils by impervious surfaces such as roofs, roads, and parking lots. The expansion of urban spaces has significantly altered natural hydrologic systems by reducing moisture retention, evapotranspiration, and groundwater recharge, and increasing peak runoff flows and discharge of toxic chemicals into surface water (McGrane 2016). Numerous studies have documented the adverse impacts associated with urban development on hydrologic systems and habitat (Jacobson 2011). These impacts include increased flooding, erosion, contaminated surface water, habitat destruction, reduced groundwater flow, and less baseflow in streams. Low-impact development (LID) has been shown to help reduce the adverse impacts of urban development (Eckart et al, 2017, Sohn et al., 2019). Many jurisdictions in the United States and Europe now require or encourage LID best management practices (BMP) when feasible. Many of these LID BMP's, including bioretention, rain gardens, grass-lined swales, infiltration trenches and ponds, pervious pavement, and drywells (also known as soak-aways), are designed to infiltrate stormwater into the subsurface.

Given the prevalence of drought conditions across much of the southwest United States, the other motivation for infiltrating stormwater to capture runoff that would otherwise flow to the ocean to use it to recharge groundwater and provide water supplies for later use. Drywells are particularly well suited for stormwater capture in urban areas because of their small footprint and the ability to install them beneath pavement. In addition, in areas with low-permeability surface soils underlain by more-permeable sand and/or gravel soils, drywells can bypass the surface soils and deliver large volumes of stormwater to the more permeable zones. Drywells have been demonstrated to be highly effective for stormwater management in the Los Angeles basin. Drywells can be constructed as dug excavations with concrete structure that extend to depths up to 6 m or as drilled wells that can extend to depths of 30 m or more with diameters of 1.2 m (Edwards et al. 2016, Sasidharan et al. 2018).

The literature review provided below summarizes infiltration test methods used for sizing shallow infiltration facilities (e.g., ponds and rain gardens) dominated by vertical flow from the facility and deep drywells dominated by horizontal flow from the facility.

B.2 Common Infiltration Test Methods Outside Los Angeles County

The Vermont agency of Natural Resources commissioned a study that summarized the methods used across the United States to design and size stormwater infiltration facilities (Stone Environmental, Inc. 2012). All of these methods utilize a design infiltration rate and most of these methods rely on soil texture, grainsize distribution, or small-scale infiltration testing to estimate the design infiltration rate. Each of these approaches offer advantages and disadvantages.

Soil Texture and Grain Size Methods

Significant research has been conducted to estimate soil hydraulic conductivity based on soil texture (Rawls et al., 1982) or grain size distribution (Hazen 1892, Carman 1956, Beyer 1964, Terzaghi and Peck 1964, Alyamani et al. 1993). Some of these methods account for the degree of grain size sorting (e.g., Beyer 1964) or porosity (Kozeny 1953, Carman 1956). These soil texture and grain size methods are often preferred because they are inexpensive and only require collection of soil samples and either visual description or grain size analysis. However, based on comparison of grain size methods and in-situ infiltration tests conducted by the author (not published) and others (Philips and Kitch 2011) the grain size methods are only useful for approximating the measured infiltration rate within an order of magnitude. In practice, bulk soil samples do not preserve the in-situ soil structure and cannot account for fine-grained layers that may significantly reduce the infiltration rate even when the fraction of fine-grained material in the bulk sample is relatively small.

Observational Infiltration Rate Methods

Most stormwater permitting jurisdictions across the United States have recognized the limitations of methods based on soil texture or grain size and either encourage or require field infiltration testing. Many of these methods include filling an open hole, a cased hole, or a ring with water and directly observing the rate of water level fall (the infiltration rate) after a soaking period. The diameter of the hole or ring is typically less than 0.6 m and the tests are relatively inexpensive to conduct and require relatively small amounts of time and water. The double-ring infiltrometer method (ASTM D3385-18, ASTM, 2018) is the most commonly recommended method of this type in the United States. The limitations of these small-scale tests have been well documented for many years (Johnson 1963, Tricker 1978, Philips and Kitch 2011). The primary issue is that these tests do not account for lateral seepage of water beneath the test hole and the measured infiltration rate decreases as the diameter of the hole becomes larger (Aronovici 1955). Jurisdictions acknowledge the limitations of these small-scale methods and require the application of correction factors that provide a design infiltration rate significantly lower than the measured infiltration rate. In addition, the amount of water used during these small-scale tests is much less than the typical storm volume passing through a full-scale infiltration facility. The influence of low-permeability perching layers below the facility are unlikely to be detected during a small-scale test (Johnson 1963) and they may have significant influence over the performance of a full-scale facility.

Robinson and Rohwer (1957) concluded that infiltration tests conducted using large-diameter rings (as much as 1.8 m for the inner ring and 5.5 m for the outer ring) provided more accurate measurements of infiltration rate than the more commonly used rings of 0.3 to 0.6 m in diameter. Obviously, there are cost and feasibility issues associated with using such large rings for typical projects. The State of Washington has recognized the importance of scale when it comes to infiltration testing and developed a more feasible large-scale test called the pilot infiltration test (PIT). This test is conducted with standing water for at least 7 hr in an excavated pit with an equivalent diameter of 1.2 to 2.0 m for the small PIT and at least 3.4 m for the large PIT. Even these larger-diameter tests do not account for lateral seepage and typically over-predict the performance of full-scale infiltration facilities. Hypothetical numerical simulations have been conducted to compare the performance of the small and large PIT results with a full-scale infiltration facility with an equivalent diameter of 36 m (Scott Kindred verbal report). The measured infiltration rate for the large PIT is 60% greater than the full-scale facility and the measured infiltration rate for the small PIT is 200% greater than the full-scale facility.

Falling-Head Borehole Permeameter Methods

As summarized by Stone Environmental, Inc. (2012), some stormwater permitting jurisdictions allow use of falling-head borehole permeameter (BP) methods that account for lateral flow, including the Philip-Dunne falling head method (Philip 1993, Muñoz-Carpena 2002), and the modified Philip-Dunne methods (Ahmed et al. 2014). These methods are more sophisticated than the methods summarized in the previous section (they account for pressure head and lateral flow) and they provide an estimate of field saturated hydraulic conductivity (K_s) rather than infiltration rate. Although many stormwater manuals use K_s and infiltration rate interchangeably and they have the same units (length/time), these concepts are significantly different. K_s is a soil property that reflects the permeability of the soil. In contrast, infiltration rate is a function of soil permeability, hydraulic gradient, soil capillarity, and the geometry of the infiltration facility. Although infiltration rate is commonly considered to represent vertical (gravity) flux out of the facility bottom, it actually measures the total flux out of the facility and includes horizontal (pressure) flux and capillary flux (capillarity). As discussed by Kindred and Reynolds (2020), the importance of pressure flux increases as the ratio of ponded head (H) versus borehole radius (r) increases and the importance of capillary flux is greater for fine-grained soils than coarse-grained soils.

Although the commonly used falling head BP methods account for lateral flow beneath the facility, they still have the same disadvantages of any small-scale method, in that they test a relatively small quantity of soil and will not detect perching layers beneath the full-scale facility. In particular, the Philip-Dunne and Modified Philip-Dunne methods only require enough water to fill the casing once (generally less than 10 L of water). Although this has been seen as an advantage since this quantify of water can be transported in small containers, this means that the water only penetrates a few centimeters into the native soil (which may be disturbed during installation of the casing) and does not address the potential groundwater mounding associated with a full-scale infiltration facility. Furthermore, the Philip-Dunne and Modified Philip-Dunne methods require the use of a driven casing and are not recommended in gravelly or dense soils due to the difficulty in driving the rings, disturbance of the in-situ soil structure, and the potential for flow-bypass up the outside of the ring.

Steady-State Borehole Permeameter Methods

As summarized by Stone Environmental, Inc. (2012), some stormwater permitting jurisdictions also allow use of uncased steady-state borehole permeameter (USSBP) methods such as the Guelph permeameter (Reynolds and Elrick1986) and the Amoozegar permeameter (Amoozegar 1992). These methods are similar to the falling-head BP methods in that they account for a more complete model of the flow dynamics and they provide an estimate of K_s. These USSBP methods are conducted in an open (uncased) borehole and continue adding water to the borehole to maintain a steady-state ponding depth until the rate of flow reaches a constant rate. K_s is calculated based on the steady-state flow rate, the ponding depth, the radius of the borehole, and the capillarity of the soil.

The USSBP methods use significantly more water than the falling-head BP methods, which generally requires a water source near the test location and can be a disadvantage. However, because the water penetrates further into the formation, the estimated K_s can provide a better representation of the performance of a full-scale infiltration facility than the falling-head BP methods. Furthermore, the steady-state BP methods do not require driving a casing and can be conducted in gravelly or dense soils with minimal disturbance of the in-situ soil structure and no potential for flow-bypass up the outside of the ring.

Traditionally, USSBP tests were conducted in relatively shallow drilled boreholes with diameters of 10 to 30 cm. Testing in these small-diameter boreholes is dominated by lateral flow out the borehole walls and are well suited for estimating the capacity of vertical infiltration facilities, such as drywells or trenches, that are dominated by horizontal flow from the facility. Large shallow infiltration facilities (such as rain gardens or shallow ponds) are dominated by vertical flow out the bottom of the facility and borehole tests may over-estimate vertical K_s due to stratigraphic layering.

B.3 County of Los Angeles Public Works Infiltration Testing Guidelines

The County of Los Angeles Public Works (CLAPW) has provided guidelines to design infiltration facilities (LA County 2021) and this guidance includes a variety of infiltration test methods, as described below:

- **Grain Size Analysis:** CLAPW guidance recommends the Hazen equation, which is subject to the same order of magnitude uncertainty as all grain size methods, as described above.

- **Double-ring infiltrometer (ASTM D3385)**. This method is described above.

- **Shallow Pit**. This test is conducted in a square or rectangular excavated pit with a flat bottom similar to the Washington State PIT method. The small version of this test has an equivalent diameter of 0.3 to 0.6 m with a ponding depth of 0.3 to 0.9 m and the large version of this test has an equivalent diameter of 0.6 to 3.0 m with a ponding depth of 0.6 to 1.8 m. This test is conducted after a pre-soaking period of up to 4 hr and may be conducted either as a falling-head test for slowly draining soils or as a steady-state test for moderate to fast draining soils. The falling-head test is conducted by refilling the hole over a period of at least 3 hr and measuring the rate of falling head, which is considered the measured infiltration rate. The steady-state test is conducted similar to the PIT test by adjusting the flow rate to maintain a steady ponding depth. However, the measured infiltration rate is calculated by dividing the flow rate by the wetted area of the pit (bottom area plus wetted sidewall area) rather than dividing the flow rate by the bottom area of the pit, as called for in the PIT method. The CLAPW shallow pit method provides lower estimates of infiltration rate that the PIT method and partially addresses the tendency of the PIT method to over-predict the infiltration rate of a full-scale infiltration facility.

- **Small Diameter Boring**. This method is conducted in a small-diameter augered boring completed as a test well. Perforated PVC casing surrounded by a filter pack is placed across the test interval and is isolated below and above with bentonite backfill in the borehole. The test interval shall correspond to the zone of infiltration for the planned infiltration facility. The infiltration test is conducted either as a falling-head or constant-head test similar to the shallow pit method and the measured infiltration rate is calculated by dividing the flow rate by the wetted area of the borehole (bottom area plus wetted sidewall area).

- **Large Diameter Boring**. This method is conducted in a large-diameter (46 to 91 cm) augered boring completed as a test well. Other than the larger diameter of the borehole, the only difference between this method and the small diameter boring method is that two PVC casings are placed in the borehole. One casing is used for introducing water and the other casing is used for measuring the water level. The infiltration test is conducted either as a falling-head or constant-head test similar to the small diameter borehole method and the measured infiltration rate is calculated by dividing the flow rate by the wetted area of the borehole (bottom area plus wetted sidewall area).

CLAPW guidance (LA County 2021) recommends that any of the methods can be used to size retention basins, bioswales, and underground galleries. Only the large diameter boring method can be used to estimate the capacity of a drywell. In theory, both the small-diameter and large diameter boring tests could be suitable for predicting the performance of drilled drywells completed over the same interval and with a similar ponding depth as the test well. These methods may not be accurate if the tested interval or the ponding depth is significantly different than the test well.

The other potential issue with the CLAPW boring test methods is that our field experience indicates that test wells drilled using hollow stem augers have provided significantly lower flow rates (i.e., orders of magnitude less) than test wells drilling using other drilling methods, such as Sonic, air rotary, and the large solid-stem augers. Hollow stem augers may have issues with smearing of the sidewalls or contamination of the filter pack.

B.4 Recommended Method for Predicting the Performance of Drywells

Outside the County of Los Angeles (LA County 2021) we are not aware of any jurisdiction in the United States providing suitable methods for predicting the performance of drilled drywells, primarily because they do not test the full vertical height of the drywell and they do not account for increased capacity as the ponding depth increases. The primary limitations of the CLAPW boring methods is that they provide an infiltration rate (rather than K_s), which is not well suitable for predicting the capacity of drywells with different diameters, sandpack intervals, or ponding depths than the test boring.

The steady-state borehole permeameter (SSBP) methods discussed in Appendix C do provide K_s and are just as easy to perform as the CLAPW boring methods. As summarized by Kindred and Reynolds (2020) the SSBP methods have evolved significantly since first developed in the 1950’s (Zanger 1953). Based on numerical simulations, Kindred and Reynolds (2020) demonstrated how the SSBP method is suitable for testing in both excavated pits and drilled test facilities with a maximum error of 13% and an average error of 3% across a broad range of test configurations and five glacially over-consolidated soil types. Although not yet published, the method has been calibrated for normally-consolidated soils typical of the Los Angeles basin with similar levels of accuracy.

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Appendix C: Cased and Uncased Steady-State Borehole Permeameter Methods

C.1 Uncased Steady-State Borehole Permeameter (USSBP) Equation

Considerable research has been conducted regarding analytical methods for estimating saturated hydraulic conductivity (K_S) from borehole infiltration tests in the unsaturated zone. These methods generally assume a flat-bottom cylindrical test facility (e.g., borehole or pit excavation), isotropic and homogeneous soil, and no water-table effects. Kindred and Reynolds (2020) provide a concise history of the evolution of this method since the 1950's. This work culminated in the mid-1980's when Reynolds et al. (1985), Reynolds and Elrick (1985), and Philip (1985) developed approximate analytical uncased steady-state borehole permeameter (USSBP) equations that formally account for pressure, gravity, and capillarity flow. The Reynolds analysis, which has been tested extensively over the years, has the form:

$$K_s = \frac{C_u Q}{2\pi H^2 + \pi r_b^2 C_u + \frac{2\pi H}{\alpha^*}} \qquad \text{(Eq. C.1)}$$

where

$$C_u = \left[\frac{(H/r_b)}{Z_1 + Z_2(H/r_b)}\right]^{Z_3} \qquad \text{(Eq. C.2)}$$

Q (m^3/d) is flow, H (m) is hydraulic head or ponding depth, r_b (m) is the borehole radius, α^* is the soil sorptive number (m^{-1}), C_u is the USSBP shape function (dimensionless), and Z_1, Z_2, and Z_3 are the shape function fitting parameters (dimensionless). Eq. C.1 assumes that H is less than the uncased or screened portion of the test facility, while other constant-head analyses assume that H is greater than the uncased or screened portion of the test facility (see e.g. Reynolds 2010). The three terms in the denominator of Eq. C.1 account, respectively, for flow through the wall and base of the test facility due to the hydrostatic pressure of the ponded water, gravity flow through the base of the test facility, and capillarity flow through the wall and base of the test facility due to the surrounding unsaturated porous material. Flow due to hydrostatic pressure accounts for most of the flow out of the test facility when $H >> r_b$, while gravity flow and capillarity flow often dominate when $H < r_b$ (Reynolds 2008; Elrick and Reynolds 1992).

Field studies have shown that α^* is relatively constant for a broad range of porous materials and can therefore be estimated using a lookup table based on soil texture and structure (Elrick et al. 1989; Reynolds 2008, 2013). Values of α^* for ten different soils were developed during an EPA-funded infiltration study (Kindred, 2022) and are listed in Table C-1. The USSBP shape function fitting parameters (Z_1, Z_2, and Z_3) were calibrated during the same study and are also provided in Table C-1. In order to provide sufficient accuracy (± 13%) four sets of USSBP $C_u(H/r)$ shape function parameters were developed to address fine-grained soil (> 12% silt), coarse-grained soil (< 12% silt), small H/r_b ratio (≤ 20), and large H/r_b ratio (≥ 20). The USSBP method is calibrated for H/r_b ratios ranging from 0.05 to 200.

C.2 Cased Steady-State Borehole Permeameter (CSSBP) Equation

The cased steady-state borehole permeameter (CSSBP) method uses a similar approach as the USSBP approach but is designed to address cased wells where the water level in the casing (H) extends higher than the sandpack interval (L). Building on work by Reynolds (2010), Kindred (2022) developed the following equation for evaluating steady-state cased infiltration tests:

$$K_s = \frac{C_c Q}{2\pi L H + \pi r_b^2 C_c + \frac{2\pi L}{\alpha^*}} \qquad \text{(Eq. C.3)}$$

Where:

$$C_c = \left[\frac{(H/r_b)}{Y_1 + Y_2(H/r_b)}\right]^{Y_3} \qquad \text{(Eq. C.4)}$$

Eq. C.3 assumes that L is less than H and is the same form as the USSBP equation (Eq. C.1) with a different shape function and some of the H's replaced by L's. The CSSBP shape function fitting parameters (Y_1, Y_2, and Y_3) were calibrated by Kindred (2022) and are provided in Table C-2. Similar to the USSBP calibration, four sets of USSBP $C_c(H/r_b)$ shape function parameters were developed to address fine-grained soil (> 12% silt), coarse-grained soil (< 12% silt), small H/r_b ratio (≤ 20), and large H/r_b ratio (≥ 20). The USSBP method is calibrated for L/r_b ratios ranging from 4 to 100.

Table C-1: *Sorptive Number* (α*) for dry/moist soil and USSBP $C_u(H/r)$ shape function (Eq. A.2) parameters (Z_1, Z_2, Z_3) for the ten representative soils. Different shape function parameters are developed for test configurations where ponded head (H) to radius (r_b) ratio was $H/r_b \leq 20$ or $H/r_b \geq 20$, and for soils with > 12% silt (USCS soil type SM) or < 12% silt.

Soil Type	**α***	**Low Ponded Head ($H/r \leq 20$)**			**High Ponded Head ($H/r \geq 20$)**		
	(ft^{-1})	**Z_1 (-)**	**Z_2 (-)**	**Z_3 (-)**	**Z_1 (-)**	**Z_2 (-)**	**Z_3 (-)**
Silty fine sand (SM)	0.50	2.11	0.192	0.91	2.04	0.0224	0.547
Silty fine-coarse sand (SM)	1.7						
Qvt (SM)	0.36						
Silty Qva (SM)	0.41						
Fine sand (SP-SM)	1.1	2.03	0.207	0.98	2.11	0.0273	0.605
Medium sand (SP)	3.4						
Sandy gravel (GW)	17.4						
Fine Qva (SP-SM)	0.76						
Fine-Medium Qva (SP)	1.2						
Fine-Coarse Qva (SW)	7.6						

Table C-2: *Sorptive Number* (α*) for dry/moist soil and C_c shape function (Eq. A.4) parameters (Y_1, Y_2, Y_3) for the ten representative soils. Different shape function parameters are developed for test configurations where screen/sandpack length (L) to borehole radius (r_b) ratio was $L/r_b < 20$ or $L/r_b \geq 20$, and for soils with > 12% silt (USCS soil type SM) or < 12% silt.

Soil Type	**α***	**Short Sandpack ($L/r_b < 20$)**			**Long Sandpack ($L/r_b \geq 20$)**		
	(ft^{-1})	**Y_1 (-)**	**Y_2 (-)**	**Y_3 (-)**	**Y_1 (-)**	**Y_2 (-)**	**Y_3 (-)**
Qvt (SM)	0.50	3.06	0.12	0.674	2.32	0.0286	0.463
Silty Qva (SM)	1.7						
Silty fine sand (SM)	0.36						
Silty fine-coarse sand (SM)	0.41						
Fine Qva (SP-SM)	1.1	2.45	0.214	0.93	1.87	0.0354	0.501
Fine sand (SP-SM)	3.4						
Fine-medium Qva (SP)	17.4						
Medium sand (SP)	0.76						
Fine-coarse Qva (SW)	1.2						
Sandy gravel (GW)	7.6						

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Appendix D: Soil Testing Results

Table D-1: Summary of B-HSA Soil Testing Results

		B-HSA, 10-11.5 ft (#3)		B-HSA, 20-21.5 ft (#5)		B-HSA, 30-31.5 ft (#8)		B-HSA, 40-41.5 ft (#12)		B-HSA, 50-51.5 ft (#16)	
Sieve Size	**Sieve Size (mm)**	**Percent Retained**	**Percent Passing**	**Percent Retained**	**Percent Passing**	**Percent Retained**	**Percent Passing**	**Percent Retained**	**Percent Passing**	**Percent Retained**	**Percent Passing**
#4	4.75	2.5	97.5	0.4	99.6	0.0	100.0	6.5	93.5	7.8	92.2
#8	2.36	14.9	85.1	0.5	99.5	0.9	99.1	13.1	86.9	15.8	84.2
#16	1.18			1.8	98.2	4.2	95.8	27.1	72.9	32.7	67.3
#30	0.6	49.4	50.6	5.1	94.9	6.6	93.4	46.8	53.2	56.4	43.6
#40	0.425	50.2	49.8					49.3	50.7		
#50	0.3			13.0	87.0	9.1	90.9			73.4	26.6
#60	0.25	50.3	49.7					63.4	36.6		
#100	0.15	50.4	49.6	20.6	79.4	13.4	86.6	68.7	31.3	79.8	20.2
#200	0.075	50.5	49.5	32.3	67.7	18.8	81.2	72.2	27.8	84.0	16.0
>#200		100.0	0.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	0.0
USCS Parameter		**B-HSA, 10-11.5 ft (#3)**		**B-HSA, 20-21.5 ft (#5)**		**B-HSA, 30-31.5 ft (#8)**		**B-HSA, 40-41.5 ft (#12)**		**B-HSA, 50-51.5 ft (#16)**	
% Gravel	>4.75 mm	2.5		0.4		0.0		6.5		7.8	
% Sand	0.075-4.75 mm	48.0		31.9		18.8		65.7		76.2	
% coarse sand	*2-4.75 mm*	*12.4*		*0.0*		*0.9*		*6.6*		*7.9*	
% medium sand	*0.425-2 mm*	*35.3*		*8.6*		*6.9*		*36.2*		*49.1*	
% fine sand	*0.074-0.425 mm*	*0.3*		*23.3*		*10.9*		*22.9*		*19.1*	
% Fines	<0.074 mm	49.5		67.7		81.2		27.8		16.0	
Soil Description (USCS)		very silty SAND, trace gravel		very sandy SILT, trace gravel		sandy SILT		silty slightly gravelly fine-med. SAND		silty slightly gravelly fine-med. SAND	
USCS Classification		SM		ML		ML		SM		SM	
Cation Exchange Capacity	meq/100g										
Total Organic Carbon	%										

Notes:

Results for #16 not considered reliable due to poor recovery.

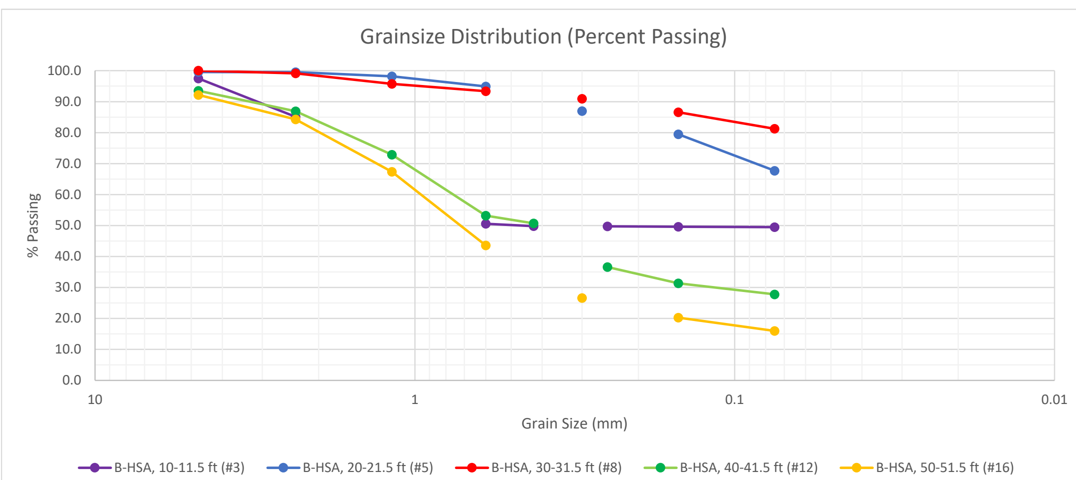


Table D-2: Summary of B-SONIC and B-Dry Soil Testing Results

		B-Dry, 15-20 ft (#8)		B-Dry, 50-55 ft (#6)		B-Dry, 55-60 ft (#2)		B-Sonic 50-60 ft (#20)		B-Sonic 60-70 ft (#21)	
Sieve Size	**Sieve Size (mm)**	**Percent Retained**	**Percent Passing**	**Percent Retained**	**Percent Passing**	**Percent Retained**	**Percent Passing**	**Percent Retained**	**Percent Passing**	**Percent Retained**	**Percent Passing**
1.5"	37.5							30.9	69.1	0.0	100.0
3/8"	9.5							30.9	69.1	6.5	93.5
#4	4.75	0.8	99.2					33.4	66.6	17.0	83.0
#8	2.36	9.8	90.2	14.6	85.4	9.4	90.6				
#10	2			19.2	80.8	11.2	88.8				
#30	0.6	48.9	51.1	49.3	50.7	61.1	38.9				
#40	0.425	50.7	49.3	59.0	41.0	78.2	21.8	61.4	38.6	67.2	32.8
#50	0.3			67.2	32.8	87.5	12.5				
#60	0.25	51.1	48.9					65.3	34.7	78.1	21.9
#100	0.15	51.6	48.4	83.8	16.2	96.5	3.5	88.5	11.5	88.4	11.6
#200	0.075	51.9	48.1	93.8	6.2	99.1	0.9	96.9	3.1	98.1	1.9
>#200		100.0	0.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	0.0
USCS Parameter		**B-Dry, 15-20 ft (#8)**		**B-Dry, 50-55 ft (#6)**		**B-Dry, 55-60 ft (#2)**		**B-Sonic 50-60 ft (#20)**		**B-Sonic 60-70 ft (#21)**	
% Gravel	>4.75 mm	0.8		0.0		0.0		33.4		17.0	
% Sand	0.075-4.75 mm	51.1		93.8		99.1		63.5		81.1	
% coarse sand	*2-4.75 mm*	*9.0*		*14.6*		*9.4*					
% medium sand	*0.425-2 mm*	*40.9*		*44.4*		*64.9*					
% fine sand	*0.074-0.425 mm*	*1.2*		*34.8*		*24.8*					
% Fines	<0.074 mm	48.1		6.2		0.9		3.1		1.9	
Soil Description (USCS)		very silty medium SAND, trace gravel		slightly silty fine-medium SAND		fine-medium SAND, trace silt		very gravelly SAND, trace silt		gravelly SAND, trace silt	
USCS Classification		SM		SP-SM		SP		SW		SW	
Cation Exchange Capacity	meq/100g										
Total Organic Carbon	%										

Notes:

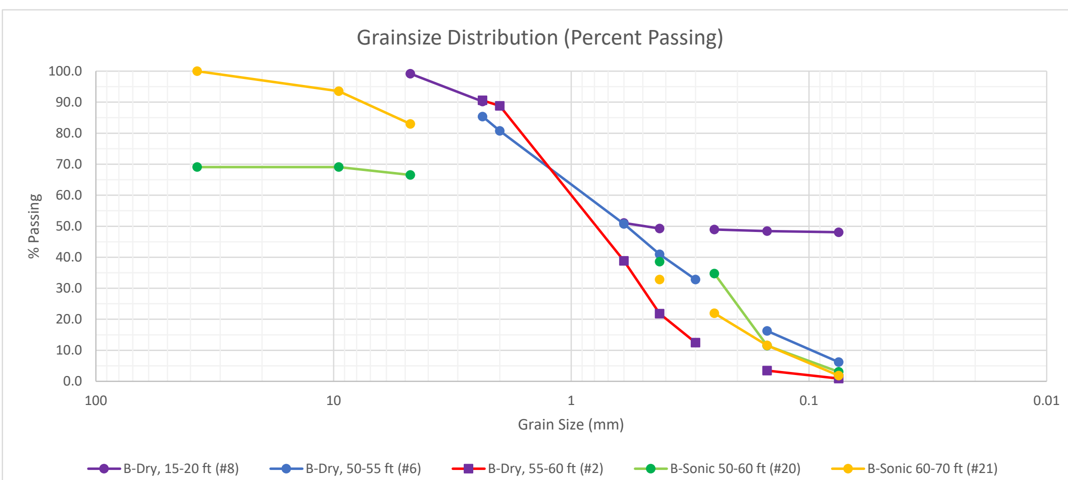
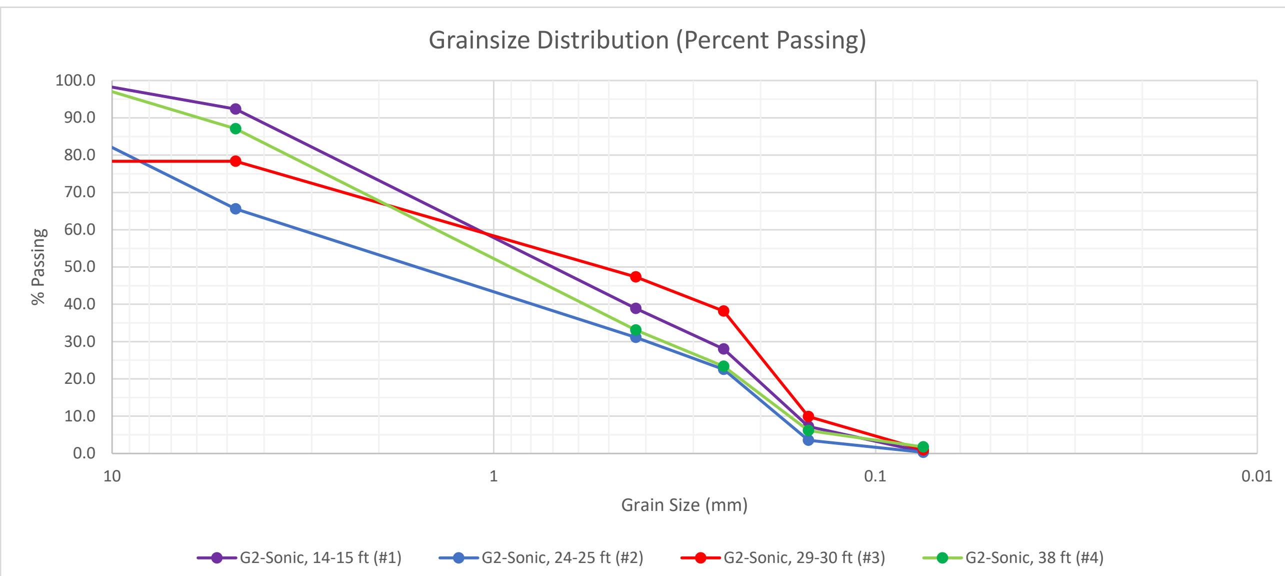


Table D-3: Summary of Glendale Soil Testing Results

		G2-Sonic, 14-15 ft (#1)		G2-Sonic, 24-25 ft (#2)		G2-Sonic, 29-30 ft (#3)		G2-Sonic, 38 ft (#4)	
Sieve Size	**Sieve Size (mm)**	**Percent Retained**	**Percent Passing**	**Percent Retained**	**Percent Passing**	**Percent Retained**	**Percent Passing**	**Percent Retained**	**Percent Passing**
3/4"	19	0.0	100.0	0.0	100.0	18.5	81.5	0.0	100.0
1/2"	12.5	0.0	100.0	13.0	87.0	21.6	78.4	0.0	100.0
#4	4.75	7.6	92.4	34.4	65.6	21.6	78.4	13.0	87.0
#30	0.425	61.1	38.9	68.9	31.1	52.6	47.4	66.9	33.1
#40	0.25	72.0	28.0	77.5	22.5	61.8	38.2	76.6	23.4
#100	0.15	92.8	7.2	96.5	3.5	90.1	9.9	93.8	6.2
#200	0.075	99.5	0.5	99.7	0.3	99.0	1.0	98.2	1.8
>#200		100.0	0.0	100.0	0.0	100.0	0.0	100.0	0.0
USCS Parameter		**G2-Sonic, 14-15 ft (#1)**		**G2-Sonic, 24-25 ft (#2)**		**G2-Sonic, 29-30 ft (#3)**		**G2-Sonic, 38 ft (#4)**	
% Gravel	>4.75 mm	7.6		34.4		21.6		13.0	
% Sand	0.075-4.75 mm	91.9		65.3		77.4		85.3	
% medium-coarse sand	*0.425-4.75 mm*	*53.5*		*34.5*		*31.0*		*54.0*	
% fine sand	*0.074-0.425 mm*	*38.4*		*30.8*		*46.4*		*31.3*	
% Fines	<0.074 mm	0.5		0.3		1.0		1.8	
Soil Description (USCS)		slightly gravelly fine-coarse SAND		very gravelly fine-coarse SAND		gravelly fine-coarse SAND, trace silt		gravelly fine-coarse SAND, trace silt	
USCS Classification		SW		SW		SW		SW	
Cation Exchange Capacity	meq/100g								
Total Organic Carbon	%								

Notes:
Percent retained for 0.05 mm size fraction interpolated between 0.063 mm and 0.032 mm assuming linear distribution between the two grain sizes.



Appendix E: Boring Logs

[(1)] Coarse-Grained Soils - More than 50% Retained on No. 200 Sieve	Gravels - More than 50% of coarse Fraction Retained on No. 4 Sieve	≤5% Fines[(4)]	GW	Well-graded gravel and gravel with sand, little to no fines
			GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥15% Fines[(4)]	GM	Silty gravel and silty gravel with sand
			GC	Clayey gravel and clayey gravel with sand
	Sands - More than 50% of Coarse Fraction Passing the #4 Sieve	≤5% Fines[(4)]	SW	Well-graded sand and sand with gravel, little to no fines
			SP	Poorly-graded sand and sand with gravel, little to no fines
		≥5% Fines[(4)]	SM	Silty sand and silty sand with gravel
			SC	Clayey sand and clayey sand with gravel
[(1)]Fine-Grained Soils - 50% or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		ML	Silt, sandy silt, gravelly silt, silt with sand or gravel
			CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		MH	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			OH	Organic clay or silt of medium to high plasticity
	Highly Organic Soils		PT	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

Coarse-Grained Soils Density	SPT[(2)] (blows/ft)	Fine-Grained Soils Consistency	SPT[(2)] (blows/ft)
Very Loose	0 to 4	Very Soft	0-2
Loose	4-10	Soft	2-4
Medium Dense	10 to 30	Medium Stiff	4-8
Dense	30-50	Stiff	8-15
Very Dense	>50	Very Stiff	15-30
		Hard	>30

Component Definitions

Descriptive Term		Size Range and Sieve Number
Boulders		Larger than 12"
Cobbles		3" to 12"
Gravel	Coarse	3" to 3/4"
	Fine	3/4" to No. 4 (4.75 mm)
Sand	Coarse	No. 4 (4.75 mm) to No. 10 (2.00 mm)
	Medium	No. 10 (2.00 mm) to No. 40 (0.425 mm)
	Fine	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay		Smaller than No. 200 (0.075 mm)

Estimated Percentage[(3)] (by weight)

Percent	Modifier	Percent	Secondary Modifier
<5	Trace	<5	Trace
5-15	Slightly (sandy, silty, clayey, gravelly)	5-10	Few
15-30	Sandy, silty, clayey, gravelly	15-25	Little
30-49	Very (sandy, silty, clayey, gravelly)	30-45	Some

Moisture Content

Dry - Absence of moisture, dusty,dry to the touch
Slightly Moist - Perceptible moisture
Moist - Damp but no visible water
Very Moist - Water visible but not free draining
Wet - Visible free water, usually from below water table

Water Levels

At time of drilling (ATD): ▽
Static water level (date): ▼
Seepage:

(1) Percentage by dry weight
(2) Standard Penetration Test (ASTM D-1586)
(3) In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
(4) Combined USCS symbols used for fines between 5% and 15% as estimated in General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
BGS = below ground surface
BOH=Bottom of hole
CAL=California Sampler

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

Kindred Hydro

Exploration Log Key

B-HSA

Project #: Drywell Testing	**Project Name**:
Location: Mary Bethune Park	**Logged by**:
Ground Surface Elevation: NAVD-88 Surveyed	**Driller**: Cascade Environmental
Exploration Method: 8-inch O.D. HSA	**Start/Finish Date:** April 13, 2023
Sampling Method: SPT	**Depth to Water (ft BGS)**: >
	Monument:None

Depth (ft)	Elev (ft)	Sample Type/I.D.	Blow Counts	Water	USCS Class	Well Const.	Description
						3-in. Solid PVC; Native Soil	**Vactor truck to 5 ft** **brown silty fine SAND**
5							
				moist	SM	Bentonite Chips	
10							
		SPT #3	10, 12	moist	SM		Medium dense, dark brown, very silty SAND, trace gravel
			10				
15							
		SPT	10, 10	moist			medium dense, brown
			12				
20							
		SPT #5	10, 14	moist	ML		very stiff, brown, very sandy SILT, trace gravel
			14				
25							
		SPT	12, 14	moist	SM		medium dense, brown, silty SAND
			14				
		SPT	14	moist			
			14, 16				
30							
		SPT #8	14, 16	moist	ML		hard, brown, sandy SILT
			18				
		SPT	16	moist			
			16, 18				
35							
		SPT	18, 18	moist			
			19				
		SPT	19	moist	SP-SM		dense, brown, slightly silty SAND with thin silt laters
			20, 20				
40							
		SPT #12	20, 21	moist			dense, brown slightly silty, gravelly, fine-medium SAND
			21				
		SPT	21	moist	CL-ML		hard, brown clayey SILT
			22, 22				
45							
		SPT	22, 23	moist	SP-SM		dense, brown silty SAND
			23			Gravel	
		SPT	23	moist	SW		dense, gravelly fine-medium SAND, trace silt
			23, 25			3-in. perforated PVC	30% recovery
50							
		SPT #16	23, 25			Cave	10% recovery
			25				
		SPT	28				50% recovery, very dense
			28, 30				
55							
		SPT	29, 29				no recovery
			30				
		SPT	28				no recovery
			28, 50/2"				
60							
		SPT	28				no recovery
			28, 50/4"				
							perforated PVC pipe wrapped with geotextile fabric

B-Sonic	Project Name:
Project #: Drywell Testing	**Logged by**:
Location: Mary Bethune Park	**Driller**: Cascade Environmental
Ground Surface Elevation: NAVD-88 Surveyed	**Start/Finish Date:** April 21, 2023
Exploration Method: 8-inch O.D. Sonic	**Depth to Water (ft BGS)**: >70
Sampling Method: Core	**Monument**:None

Depth (ft)	Elev (ft)	Sample Type/I.D.	Blow Counts	Water	USCS Class	Well Const.	Description
						Native Soil	Not logged
5							
10							
15							
20						2-in. Solid PVC; Bentonite Chips	
25							
30							
35							
40							
45							
50					SW		very gravelly to gravelly SAND, trace silt
55		Core #20				2-in. slotted PVC; sand pack	
60							
65		Core #21				Bentonite Chips	
70							

B-Dry	Project Name:
Project #: Drywell Testing	**Logged by**:
Location: Mary Bethune Park	**Driller**: Torrent Resources
Ground Surface Elevation: NAVD-88 Surveyed	**Start/Finish Date:** April 11, 2023
Exploration Method: 48-inch diameter solid stem auger	**Depth to Water (ft BGS)**: >60
Sampling Method: grab	**Monument**:None

Depth (ft)	Elev (ft)	Sample Type/I.D.	Blow Counts	Water	USCS Class	Well Const.	Description
					SM/SP-SM	Native Soil	soil discription and depths are uncertain due to drilling method
							dk brown, moist, very silty to silty fine-medium SAND
5							
10						6-in. Solid PVC; Bentonite Chips in bottom 1.3 ft, bentonite grout to 5 ft below surface	
15							
		Grab #8					very silty medium SAND, trace gravel
20							
							clay layer at 24 ft
25							
30							
35							silty fne SAND
40							
45							
					SW		fine-medium SAND trace silt, occational gravel
50						6-in. slotted PVC; gravel pack	
		Grab #6					gravelly, slightly silty fine-medium SAND
55							
		Grab #2					gravelly fine-medium SAND, trace silt
							geotextile fabric lining hole down to a depth of 50 ft
60							1-inch piezometer installed to bottom of hole at 61 ft with 5-ft of slotted screen

G2-Sonic		**Project Name**:
Project #: Drywell Testing		**Logged by**:
Location: Glendale Site 2		**Driller**: Cascade Environmental
Ground Surface Elevation:	NAVD-88 Surveyed	**Start/Finish Date:** June 1, 2023
Exploration Method: 8-inch O.D. Sonic		**Depth to Water (ft BGS)**: >55
Sampling Method: Core		**Monument**:None

Depth (ft)	Elev (ft)	Sample Type/I.D.	Blow Counts	Water	USCS Class	Well Const.	Description
							Vactor truck to 8'
				moist	SM/SM-SP	Bentonite Chips Native Soil	dark brown, moist, slightly silty to silty fine SAND with occational cobbles
5						2-in. Solid	
10							
				moist	SM		dark brown, moist, very silty fine SAND
				slightly moist	SP/SM/SM		tan brown, slightly moist, slightly silty, slightly gravelly fine-coarse SAND, layers of very silty SAND
		Core #1					
15							
					ML		tan brown, slightly moist, slightly sandy SILT
20				slightly moist			
					SP-SM		tan brown, slightly moist, slightly silty fine-coarse SAND with ~15% gravel
		Core #2			SP		gravelly to very gravelly, fine-coarse SAND, trace silt
25							
		Core #3					
30						2-in. slotted PVC sand pack	
35							
				moist			
		Core #4					
40							
				moist	SP		fine-medium SAND, trace gravel, trace silt
45							
							4" thick silty SAND layer
50						Bentonite Chips	
				moist	SM		silty fine-medium SAND
55							

G2-HSA

Project #: Drywell Testing	**Project Name**:
Location: Glendale Site 2	**Logged by**:
Ground Surface Elevation: NAVD-88 Surveyed	**Driller**: Cascade Environmental
Exploration Method: 8-inch HSA	**Start/Finish Date:** June 1, 2023
Sampling Method: SPT	**Depth to Water (ft BGS)**: >45
	Monument:None

Depth (ft)	Elev (ft)	Sample Type/I.D.	Blow Counts	Water	USCS Class	Well Const.	Description
							Vactor truck to 8'
					SM/SM-SP	2-in. Solid; Native Soil	dark brown, moist, slightly silty to silty fine SAND with occational cobbles
5							
10					SM	Bentonite Chips	dark brown, moist, very silty fine SAND
15		SPT			SP		dark brown, slightly moist, slightly gravelly, fine-medium SAND, trace silt
20		SPT			SP-SM		brown, slightly silty fine SAND, trace gravel
25		SPT			SP		brown fine-coarse SAND, trace silt
30		SPT					brown, fine-medium SAND, trace silt, trace gravel
35		SPT				2-in. slotted PVC; sand pack	
40		SPT					
45		SPT					transition between soil texture uncertain due to 5 ft sampling

Note: Discriptions may underestimate amount of gravel due to poor recovery.