APPENDIX A

Stormwater Management Practice

Hierarchy Checklist

SMP HIERARCHY CHECKLIST - CSS AREAS

Tier[c]	Function Type[d]	Practice Type[e]	Percent of SMP volume applied[a]: WQv	RRv	Vv	Site constraints that limit SMP feasibility[b]: Soil	Subsurface	Hotspot	Surfaces	Space
Tier 1	Infiltration (Vegetated)	Bioretention	100	100	50	×	×	×	×	×
		Rain garden	100	100	50	×	×	×	×	×
		Stormwater planter	100	100	50	×	×	×	×	×
		Tree planting / preservation	SC	SC	0					
		Dry basin	100	100	50	×	×	×	×	×
		Grass filter strip	SC	SC	0	×	×	×	×	×
		Vegetated swale	SC	SC	0	×	×	×	×	×
	Evapotranspiration[f]	Rain garden	100	100	0		×		×	×
		Stormwater planter	100	100	0				×	
		Tree planting / preservation	SC	SC	0					
		Green roof	100	100	0					
Tier 2	Infiltration (Non-vegetated)	Dry well	100	100	50	×	×	×		×
		Stormwater gallery	100	100	50	×	×	×		×
		Stone trench	100	100	50	×	×	×	×	×
		Porous pavement	100	100	50	×	×	×		×
		Synthetic turf field	100	100	50	×	×	×	×	×
Anytime / Optional	Reuse	Rain tank	100	100	SC					
		Cistern	100	100	SC					
Tier 3	Detention[g,h,i]	Dry basin	100	0	100		×		×	×
		Constructed wetland	100	0	100		×		×	×
		Wet basin / pond	100	0	100		×		×	×
		Stormwater gallery	100	0	100		×			×
		Blue roof	100	0	100					
		Detention tank	100	0	100					

[a]Values marked "SC" are special cases for criteria-based practices, see Section 4.11 for details on criteria and application.

[b]An "X" marker indicates the site constraints that would prevent each practice from being used, contingent on the appropriate documention for that constraint.

[c]All practices of higher tiers must be used to the maximum extent possible or eliminated due to site constraints, before moving to lower tier practices

[d]Details on the design criteria and applied volumes for dual function systems are available in Section 4.9 on Innovative Systems.

[e]Other practice types not shown here may be proposed, subject to DEP approval, see Section 4.9 on Innovative Systems.

[f]Where permeablity rates of the site are 0.5 in/hr or greater, rain gardens, stormwater planters, and tree planting/preservation must be designed as infiltration practices

[g]High groundwater (subsurface constraint) limits the use of most practices, except those enclosed in concrete with adequate anchoring, as determined by an engineer

[h]Detention practices may be used to manage WQv in CSS areas when the release rate complies with the sewer operations requirement (i.e., 0.1 cfs/acre)

[i]Detention practices in series (e.g., blue roof to detention tank) require special calculations to account for changes in required detention volumes

SMP HIERARCHY CHECKLIST - MS4 AREAS

Tier[c]	Function Type[d]	Practice Type[e]	WQv	RRv	Vv	Soil	Subsurface	Hotspot	Surfaces	Space
			Percent of SMP volume applied[a]			Site constraints that limit SMP feasibility[b]				
Tier 1	Infiltration (Vegetated)	Bioretention	100	100	50	×	×	×	×	×
		Rain garden	100	100	50	×	×	×	×	×
		Stormwater planter	100	100	50	×	×	×	×	×
		Tree planting / preservation	SC	SC	0					
		Dry basin	100	100	50	×	×	×	×	×
		Grass filter strip	SC	SC	0	×	×	×	×	×
		Vegetated swale	SC	SC	0	×	×	×	×	×
	Evapotranspiration[f]	Rain garden	100	100	0		×		×	×
		Stormwater planter	100	100	0				×	
		Tree planting / preservation	SC	SC	0					
		Green roof	100	100	0					
Tier 2	Infiltration (Non-vegetated)	Dry well	100	100	50	×	×	×		×
		Stormwater gallery	100	100	50	×	×	×		×
		Stone trench	100	100	50	×	×	×	×	×
		Porous pavement	100	100	50	×	×	×		×
		Synthetic turf field	100	100	50	×	×	×	×	×
Anytime / Optional	Reuse	Rain tank	100	100	SC					
		Cistern	100	100	SC					
Tier 3	Filtration[g]	Bioretention	100	40	0		×		×	×
		Stormwater planter	100	40	0		×		×	×
		Porous pavement	100	0	0		×			×
		Synthetic turf field	100	0	0		×		×	×
		Sand filter	100	0	0		×		×	
		Organic filter	100	0	0		×		×	
	Detention[g,h]	Constructed wetland	100	0	100		×		×	×
		Wet basin / pond	100	0	100		×		×	×
Other	Detention[g,i,j]	Dry basin	0	0	100		×		×	×
		Stormwater gallery	0	0	100		×			×
		Blue roof	0	0	100					
		Detention tank	0	0	100					

[a] Values marked "SC" are special cases for criteria-based practices, see Section 4.11 for details on criteria and application.
[b] An "X" marker indicates the site constraints that would prevent each practice from being used, contingent on the appropriate documention for that constraint.
[c] All practices of higher tiers must be used to the maximum extent possible or eliminated due to site constraints, before moving to lower tier practices
[d] Details on the design criteria and applied volumes for dual function systems are available in Section 4.9 on Innovative Systems.
[e] Other practice types not shown here may be proposed, subject to DEP approval, see Section 4.9 on Innovative Systems.
[f] Where permeablity rates of the site are 0.5 in/hr or greater, rain gardens, stormwater planters, and tree planting/preservation must be designed as infiltration practices
[g] High groundwater (subsurface constraint) limits the use of most practices, except those enclosed in concrete with adequate anchoring, as determined by an engineer
[h] Select detention practices with treatment abilities may be used to manage WQv in MS4 areas when all design criteria are met
[i] Remaining detention practices may only be used to meet sewer operations criteria, included here for completeness
[j] Detention in series (e.g., blue roof to detention tank) require special calculations to account for changes in required detention volumes

APPENDIX B

Nitrogen No-Net-Increase Calculator Guide

NYC MS4 No-Net-Increase Calculator for Nitrogen

Non-negligible land use changes can increase the amount of nitrogen within stormwater runoff. This increase can be calculated by comparing the existing site conditions before a project has begun (pre-construction) and after a project is completed (post-construction). The simplified procedures for using DEP's interactive tool, the NYC MS4 No-Net-Increase Calculator for Nitrogen, are described below.

DEP developed the NYC MS4 No-Net-Increase Calculator for Nitrogen to aid applicants in demonstrating NNI of nitrogen resulting from a project subject to NNI requirements. The calculator compares existing site conditions (pre-construction) to post-construction conditions and outputs the net change in nitrogen loads based on the calculated WQv.

Overview of Calculator

The NYC MS4 No-Net-Increase Calculator for Nitrogen input and output page is shown in Figure 3-4. The online version of the calculator is located on the DEP MS4 web page (https://www1.nyc.gov/assets/dep/downloads/pdf/water/stormwater/ms4/nni-calculator.xlsx).

Figure 3-4. NYC MS4 No-Net-Increase Calculator for Nitrogen

NYC MS4 No-Net-Increase Calculator

NYC Environmental Protection

Project Name:	[Enter Name]	Prepared For:	[Enter Owner Name]
DEP Application Number:	[Enter Number]	Prepared By:	[Enter Company Name]
Borough; Block, and Lot:	[Enter Borough; Block #, Lot #]	Date:	[Enter Date]

Step 1: Nitrogen Load Calculation (DRAFT)

This section calculates the change in nitrogen load from pre- to post-construction site conditions (see Nitrogen Load Calculation tab). Please fill in shaded cells.
Any increase in nitrogen load must be removed using stormwater management practices (SMPs).

Pre-Construction			Post-Construction		
Project Area (acres)			Project Area (acres)		
Impervious Area (acres)			Impervious Area (acres)		
Current Land Use			Proposed Land Use		
Runoff Coefficient (R_v)			Runoff Coefficient (R_v)		
Total Nitrogen Load (Pre)		lbs	Total Nitrogen Load (Post)		lbs
			Required Nitrogen Load Reduction		lbs
			Percent Reduction Required		%

Step 2: SMP Nitrogen Removal Calculation (DRAFT)

This section calculates the nitrogen load reduction for proposed SMPs. Load reduction calculation considers both pervious and impervious areas within SMP catchment area.
Fill in shaded cells for post-construction conditions. Use a separate row for each catchment area draining to an SMP. SMP must be sized to manage the entire SMP catchment area.
For alternative SMPs not in drop down (manufactured technologies or treatment trains), see NYC SWDM and enter SMP type and removal rate in Rows 7-10 (must attach documentation).

	SMP Catchment Area (acres)	Impervious Area (acres)	SMP Type	Total Nitrogen Removal Rate (%)	Total Nitrogen Load Reduction (lbs)
1					
2					
3					
4					
5					
6					
7			[Enter Other SMP Type]		
8			[Enter Other SMP Type]		
9			[Enter Other SMP Type]		
10			[Enter Other SMP Type]		
	0.0	0.0			0.00

Step 3: No-Net Increase Verification (DRAFT)

This section verifies that proposed SMPs will reduce the post-construction nitrogen load equal to or less than the pre-construction nitrogen load, resulting in no net increase.

	Load (lbs)	Percent (%)	
Required Nitrogen Load Reduction			(from Step 1)
Actual Nitrogen Load Reduction			(from Step 2)

PLEASE COMPLETE STEPS 1 AND 2

The TN load change is calculated by subtracting the pre-construction TN load from the post-construction TN load, using the equation below. The TN load for pre- and post-construction conditions is determined by multiplying the water quality volume (WQv) for the project area by the event mean concentration (EMC) for TN for its associated land use type, as per Table 3-1. The WQv is found using the formula from Chapter 4 of the NYS SWMDM, with a minimum value for the volumetric runoff coefficient Rv of 0.2.

WQv (post) * EMCTN (post) – WQv (pre) * EMCTN (pre) = TN load change

If the post-construction load is greater than the pre-construction load, the calculated value for the net increase serves as the basis for the stormwater management recommendations and should be included in the SWPPP. Any resulting net TN load increase must be removed using appropriately selected and designed SMPs, detailed in Table 3-2.

Accounting for Pervious and Impervious Area Conditions

Increasing pervious surface area onsite may help to avoid NNI requirements all together (see definition of "Negligible Land Use Change"). DEP encourages developers to increase pervious areas in the post-construction site condition during site planning, to the greatest extent possible. DEP considers green roofs, porous pavement, vegetated SMPs, or other landscaped pervious areas for the purpose of calculating WQv and required nitrogen load reduction in Step 1. In addition, TN removal in stormwater runoff from impervious and pervious surfaces managed by various SMPs is determined in Step 2 of the calculator as shown in Table 3-2.

Event Mean Concentrations of TN

Table 3-1 shows median values for TN EMCs for common land uses in NYC, related zoning districts, and similar or applicable land uses included in the NYSDEC Notice of Intent (NOI) form. The values in Table 3-1 were derived by comparing estimated EMCs for various land use types across 10 national studies. The NYC MS4 No-Net-Increase Calculator for Nitrogen uses the values from this table as land use loading coefficients when computing TN loadings for the project area.

Table 3-1. Median EMCs for TN

NYC Land Use	NYC Zoning Districts	Similar or Applicable Land Uses From NOI	EMC for TN (mg/L)
Commercial	C1-C8	Institutional/School, Municipal	2.08
Industrial/Manufacturing	M1-M3	Linear Utility, Well Drilling Activity (Oil, Gas, etc.), Road/ Highway, Parking Lot	2.10
Vacant/Open Space	NA	Forest, Pasture/Open Land, Cultivated Land, Recreational/ Sports Field, Bike Path/Trail, Clearing/Grading, Demolition/No Redevelopment	1.50
Lower-Density Residential	R1-R5	Single Family Home/Subdivision	2.10
Moderate- and Higher-Density Residential	R6-R10	Town Home Residential, Multifamily Residential	2.41

Note: mg/L = milligrams per liter.

User Inputs

For the NYC MS4 No-Net-Increase Calculator for Nitrogen, the SWPPP preparer will be responsible for inputting the following information:

- Total project area (acres)
- Pre-construction conditions for the total project area
  - » Impervious area (acres)
  - » Current land use type (from dropdown menu)
- Post-construction conditions for the total project area
  - » Impervious area (acres)
  - » Proposed land use type (from dropdown menu)

Calculator Outputs

Post-construction TN load will depend on land use changes and the EMCs for these land use types, as well as impervious cover changes. The calculator will compare the pre- and post-construction conditions and output the resulting net changes in TN load, as a quantity in pounds (lbs) and percentage (%).

DEP recommends reducing the post-construction impervious area to the greatest extent feasible, to mitigate stormwater runoff increases and net increases in TN load. As a next step toward compliance with NNI requirements, SMPs described in Table 3-2, must be implemented in the SWPPP to remove all net increases in TN load from the covered development project.

Post-construction conditions at a 1.7-acre site in the Jamaica Bay watershed.

SMPs for Nitrogen Removal

For projects subject to NNI requirements which drain to nitrogen-impaired receiving waterbodies, SWPPP preparers must implement SMPs to mitigate any net increases in nitrogen due to non-negligible land use changes. Table 3-2 is a list of pollutant removal rates by SMP. DEP derived these values by comparing SMP TN removal rate data from a number of different national research reports, regional design documents, and state and municipal manuals. The third column refers to the appropriate guidance in the NYS SWMDM for each SMP. However, SWPPP preparers should refer to all applicable sections in Chapters 5, 6, and 7 of the NYS SWMDM for SMP design and selection information.

Table 3-2. TN Removal by SMP

SMP	TN Removal Rate	NYS SWMDM Section
Rainwater Reuse System	100%	Section 5.3.10
Rain Garden	100%	Section 5.3.7
Bioretention	100%	Section 6.4
Porous Pavement	100%	Section 5.3.11
Infiltration Trench	100%	Section 6.3
Turf Field	40%	N/A
Sand Filter (Filtration)	40%	Section 6.4
Bioretention with Underdrain	40%	Section 6.4
Porous Pavement with Underdrain	40%	Section 5.3.11
Green Roof	35%	Section 5.3.8
Constructed Wetlands	35%	Section 6.2
Ponds	30%	Section 6.2

SMPs should be selected based on site conditions such as infiltration feasibility, available space, land use, soil suitability, site slope, depth to groundwater, and O&M requirements. The catchment areas draining to individual SMPs (or SMPs in series, as described below) need to be delineated accurately and included in the calculator to assess the overall pollutant load reduction for the entire project area.

The NYC MS4 No-Net-Increase Calculator for Nitrogen allows applicants to assign the TN removal rates in Table 3-2 to each SMP catchment area based on the selection and design of corresponding SMPs. The calculator estimates the total removal efficiencies across all SMP catchment areas and compares the TN removed by the SMPs to the net TN increase due to the development activity. The total post-construction TN load for the project area must be less than or equal to the total pre-construction TN loads. All NNI calculations for TN must be included and documented in the SWPPP. An example NYC MS4 No-Net-Increase Calculator for Nitrogen calculation is provided in Attachment 2.

Treatment Trains and Manufactured Technologies for Nitrogen Removal

SWPPP preparers may use alternative technologies not listed in Table 3-2 to achieve TN NNI requirements. SWPPPs that propose alternative technologies must include supporting documentation to verify TN removal efficiencies.

DEP will rely on the approval processes referenced in Chapter 3 of the NYS SWMDM, including the requirement that the alternative technology must be approved by a third party verification program (https://www.dec.ny.gov/chemical/29089.html).

For alternative technologies, including proprietary water quality treatment devices that are not included in or do not meet the standards of the NYS SWMDM, supporting documentation of TN removal rates must follow the approach currently employed by NYSDEC to verify technology effectiveness. Specifically, applicants must provide evidence of third party verification from Washington State's Technology Assistance Protocol - Ecology (TAPE) Program or the multi-state Technology Acceptance Reciprocity Partnership (TARP) Program for TN removal rates applied for each proposed alternative technology in the calculator.

SWPPP preparers may also elect to implement multiple SMPs in series, referred to as a treatment train, to treat runoff from the same SMP catchment area and achieve NNI requirements for the project area. This can be an effective way to achieve NNI requirements for sites where a single SMP for each catchment area cannot achieve the required TN load reduction, or for space-constrained sites.

For example, rooftop runoff can be treated with a green roof and outflow from the green roof can then be discharged to a sand filter or other approved treatment technology at ground level. With this post-construction condition, TN load is effectively reduced first through the green roof and remaining load is reduced further by the sand filter. In order for a treatment train to be effective, the SMPs utilized must be different types of technologies (i.e. placing two sand filters in a row is not considered a treatment train). Figure 3-5 represents a schematic of a treatment train with three different SMPs implemented in series.

SWPPP preparers should use the calculation below to identify the TN removal rate of an SMP treatment train for a specific SMP catchment area:

$$Rr = [1 - ((1 - rr1)*(1 - rr2)*(1 - rr3))] * 100$$

Where:

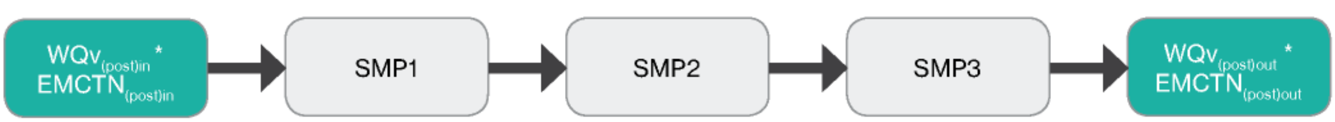
Rr = overall removal rate (%)

rr1, 2, 3 = removal rates for SMP1, SMP2, and SMP3, respectively (%)

The TN load of the inflow is first treated by SMP1 with a TN removal efficiency of rr1 (removal rate for SMP1), and the remainder pollutant load is then treated by SMP2 with a removal efficiency of rr2 (removal rate for SMP2), and so on.

The calculation for each SMP catchment area with a proposed treatment train needs to be provided as supporting documentation with the SWPPP. Removal rates in Table 3-2 should be used for each SMP proposed in series or, if an alternative technology is proposed, the guidance below should be used. The overall removal rate (Rr) calculated should be entered into the NYC MS4 No-Net-Increase Calculator as the TN removal rate for an SMP treatment train to demonstrate that NNI requirements are met.

Figure 3-5. SMP Treatment Train Schematic



NYC MS4 No-Net-Increase Calculator for Nitrogen - Example

In this example, proposed redevelopment activities will increase the impervious area on a 4.0-acre site in the Flushing Bay watershed by 0.5 acres, which will trigger NNI requirements, Figure 1.

Figure 1 - Four-acre site in Flushing Bay watershed with proposed increase in impervious surfaces that must meet NNI requirements.

The NYC MS4 No-Net-Increase Calculator input table for the project site in Figure 1 is presented in Figure 2.

Figure 2 – NYC MS4 No-Net-Increase Calculator for the Four-acre site in Flushing Bay watershed with proposed increase in impervious surfaces that must meet NNI requirements.

NYC MS4 No-Net-Increase Calculator

Project Name:	*Four-Acre Example*	Prepared For:	*[Enter Owner Name]*
DEP Application Number:	*[Enter Number]*	Prepared By:	*[Enter Company Name]*
Borough; Block, and Lot:	*[Enter BBL]*	Date:	*[Enter Date]*

Step 1: Nitrogen Load Calculation (DRAFT)

This section calculates the change in nitrogen load from pre- to post-construction site conditions (see Nitrogen Load Calculation tab). Please fill in shaded cells.
Any increase in nitrogen load must be removed using stormwater management practices (SMPs).

Pre-Construction	
Project Area (acres)	4.00
Impervious Area (acres)	2.50
Current Land Use	Commercial
Runoff Coefficient (R_v)	0.61
Total Nitrogen Load (Pre)	1.73 lbs

Post-Construction	
Project Area (acres)	4.00
Impervious Area (acres)	3.00
Proposed Land Use	Commercial
Runoff Coefficient (R_v)	0.73
Total Nitrogen Load (Post)	2.05 lbs
Required Nitrogen Load Reduction	**0.32** lbs
Percent Reduction Required	**16%**

Step 2: SMP Nitrogen Removal Calculation (DRAFT)

This section calculates the nitrogen load reduction for proposed SMPs. Load reduction calculation considers both pervious and impervious areas within SMP catchment area.
Fill in shaded cells for post-construction conditions. Use a separate row for each catchment area draining to an SMP. SMP must be sized to manage the entire SMP catchment area.
For alternative SMPs not in drop down (manufactured technologies or treatment trains), see NYC SWDM and enter SMP type and removal rate in Rows 7-10 (must attach documentation).

SMP Catchment Area (acres)	Impervious Area (acres)	SMP Type	Total Nitrogen Removal Rate (%)	Total Nitrogen Load Reduction (lbs)
1.00	0.00	Green Roof	35%	0.05
1.00	1.00	Sand Filter (Filtration)	40%	0.27
		[Enter Other SMP Type]		
		[Enter Other SMP Type]		
		[Enter Other SMP Type]		
		[Enter Other SMP Type]		
2.00	1.00			0.32

Step 3: No-Net Increase Verification (DRAFT)

This section verifies that proposed SMPs will reduce the post-construction nitrogen load equal to or less than the pre-construction nitrogen load, resulting in no net increase.

	Load (lbs)	Percent (%)	
Required Nitrogen Load Reduction	**0.32**	**16%**	(from Step 1)
Actual Nitrogen Load Reduction	**0.32**	**16%**	(from Step 2)

NO-NET-INCREASE REQUIREMENTS MET

As shown in Figure 2, the pre- and post-development conditions for the inputs for Step 1: Nitrogen Load Calculation are below, together with the calculated total nitrogen load:

Pre-Construction:

- Project Area: 4.0 acres
- Impervious Area: 2.5 acres
- Current Land Use: Commercial
- Total Nitrogen Load (pre): 1.73 lbs.

Post-Construction:

- Project Area: 4.0 acres
- Impervious Area: 3.0 acres
- Proposed Land Use: Commercial
- Total Nitrogen Load (post): 2.05 lbs.

Note that the pervious surface area of green roofs, porous pavement, vegetated SMPs, or other landscaped areas should not be included in the impervious area cell under Step 1 or Step 2. In this example, a green roof is considered pervious area not impervious area and, consequently, the WQv and required nitrogen load reduction is less than if considered a regular roof. The green roof also provides limited nitrogen removal in Step 2 given a minimum runoff coefficient of 0.2 for all surfaces (impervious and pervious).

Therefore, in this example, the SWPPP preparer is required to install SMPs to remove 0.32 lbs. (or 16%) of total nitrogen, which represents the load increase between pre- and post-development.

The SWPPP preparer proposes multiple SMPs and enters their associated catchment areas into the upper rows of the table in Step 2: SMP Nitrogen Removal Calculation. The calculator assigns the appropriate nitrogen removal rates and identifies the total nitrogen load removed per SMP.

SMP 1 Type: Green Roof

Impervious Area (First SMP Catchment Area): 0.0 acres

Total Nitrogen Removal Rate: 35%

Total Nitrogen Load Reduction: 0.05 lbs.

SMP 2 Type: Porous Pavement

Impervious Area (Second SMP Catchment Area): 1.0 acre

Total Nitrogen Removal Rate: 40%

Total Nitrogen Load Reduction: 0.27 lbs.

The total nitrogen load removal for the proposed SMPs is 0.32 lbs. (or 16%), which equals the NNI requirements as verified in Step 3: No-Net Increase Verification. The developer should print the calculator results as confirmation and include it in their SWPPP submittal.

APPENDIX C

Stormwater Management Practice

Siting Criteria

SMP Siting Criteria

HORIZONTAL SETBACKS	Minimum Setback Distance (feet)
Building Foundations, Vaults and Protruded Basements	10
Flagpoles and Light Poles	10
Retaining Walls	10
Transit Structures	25
Highway/Roadway Structures	25
Monitoring Wells	50
DEP Infrastructure (e.g. water and/or sewer mains, etc.)	15
Property Line	5
Slopes 10% below practice Slopes 10% - 30% below the practice Note: avoid installing an infiltration facility near slopes greater than 30%.	100 100 + 5 feet for every 1% slope
VERTICAL SEPARATION	
Bottom of practice to the top of the high groundwater table	3
Bottom of practice to the top of bedrock or other impermeable material or subsurface layer	3

APPENDIX D

Stormwater Management Practice Sizing Examples

WATER QUALITY VOLUME SIZING EXAMPLES

Infiltration (vegetated)

Stormwater Planter

Design a stormwater planter that will treat the water quality volume from an impervious area of 3,000 square feet, with a runoff coefficient of 0.95. Assume a media saturated hydraulic conductivity of 2 in/hr and an infiltration rate of 2 in/hr.

Step 1: Calculate the WQ_V.

$$WQ_V = \frac{1.5\ in}{12} * A * \ R_V$$

where:
WQ_V = water quality volume (cf)
A = contributing area (sf) = 3,000 sf
R_V = runoff coefficient relating total rainfall and runoff
R_V = 0.05 + 0.009(I) = 0.95
I = percent impervious cover = 100%

$$WQ_V = \frac{1.5\ in}{12} * 3{,}000\ sf * \ 0.95$$

$$WQ_V = 356.25\ cf$$

Step 2: Calculate the SMP area assuming a maximum loading ratio of 1:20 for a stormwater planter practice. Use the area to set the initial length and width of the practice.

$$A_{SMP} = \frac{A}{20}$$

where:
A_{SMP} = area at the base of infiltration SMP (sf)
A = contributing area (sf) = 3,000 sf

$$A_{SMP} = \frac{3{,}000\ sf}{20}$$

$$A_{SMP} = 150\ sf$$

Assume a 15 ft by 10 ft practice.

Step 3: Calculate the volume of surface ponding assuming a surface ponding depth of 0.5 ft, which is less than the maximum surface ponding depth of 1 ft for a stormwater planter practice.

$$V_P = A_{SMP} * D_P$$

where:

V_P = volume of surface ponding (cf)
A_{SMP} = area of the SMP (sf) = 150 sf
D_P = depth of ponding (ft) = 0.5 ft

$$V_P = 150\ sf * 0.5\ ft$$

$$V_P = 75\ cf$$

In this case, the designer has chosen to use a hydraulic connection between the ponding zone and the stone base. Therefore, the ponding zone does not need to temporarily store 75% of the water quality volume.

Step 4: Calculate the volume of voids in the soil media layer assuming a soil media depth of 1.5 ft equal to the minimum soil media depth of 1.5 ft for a stormwater planter practice.

$$V_S = A_{SMP} * D_S * n_S$$

V_S = volume of voids in the soil media layer (cf)
A_{SMP} = area of the SMP (sf) = 150 sf
D_S = depth of soil media layer (ft) = 1.5 ft
n_S = available porosity of soil media (cf/cf) = 0.2 cf/cf

$$V_S = 150\ sf * 1.5\ ft\ * 0.2 \frac{cf}{cf}$$

$$V_S = 45\ cf$$

Step 5: Calculate the volume of voids created by internal structures.

Assume there are no internal structures in this stormwater planter practice, so the volume is 0.

$$V_I = 0\ cf$$

Step 6: Calculate the volume of voids in the drainage layer assuming a drainage media depth of 1 ft, which is equal to the minimum drainage media depth of 1 ft for a stormwater planter practice.

$$V_D = (A_{SMP} * D_D - V_{I,d}) * n_D$$

where:
V_D = volume of voids in the drainage layer (cf)
A_{SMP} = area of the SMP (sf) = 150 cf
D_D = depth of the drainage layer (ft) = 1 ft
$V_{I,d}$ = volume of voids created by internal structures within the drainage layer (cf) = 0 cf
n_D = porosity of drainage layer media (cf/cf) = 0.4 cf/cf

$$V_D = (150\ sf * 1\ ft - 0\ cf) * 0.4 \frac{cf}{cf}$$

$$V_D = 60\ cf$$

Step 7: Calculate the total SMP volume from the individual component volumes and compare to the WQ_V.

$$V_{SMP} = V_P + V_S + V_I + V_D$$

where:
V_{SMP} = storage volume of SMP (cf)
V_P = volume of surface ponding (cf) = 150 cf
V_S = volume of voids in the soil media layer (cf) = 90 cf
V_I = volume of voids created by internal structures such as chambers or pipes (cf) = 0 cf
V_D = volume of voids in the drainage layer (cf) = 120 cf

$$V_{SMP} = 150\ cf + 45\ cf + 0\ cf + 60\ cf$$

$$V_{SMP} = 255\ cf\ < WQ_V = 356.25\ cf \qquad NO$$

Practice does not manage the entire WQv. Reconfigure the practice to increase the storage volume and return to associated step. In this case, the practice area will be increased, and Steps 2-8 are repeated.

Step 2: Calculate the SMP area assuming a loading ratio of 1:10, which is less than the maximum loading ratio of 1:20 for a stormwater planter practice. Use the area to set the initial length and width of the practice.

$$A_{SMP} = \frac{A}{10}$$

where:
A_{SMP} = area at the base of infiltration SMP (sf)
A = contributing area (sf) = 3,000 sf

$$A_{SMP} = \frac{3{,}000\ sf}{10}$$

$$A_{SMP} = 300\ sf$$

Assume a 30 ft by 10 ft practice.

Step 3: Calculate the volume of surface ponding assuming a surface ponding depth of 0.5 ft, which is less than the maximum surface ponding depth of 1 ft for a stormwater planter practice.

$$V_P = A_{SMP} * D_P$$

where:
V_P = volume of surface ponding (cf)
A_{SMP} = area of the SMP (sf) = 300 sf
D_P = depth of ponding (ft) = 0.5 ft

$$V_P = 300\ sf * 0.5\ ft$$

$$V_P = 150\ cf$$

In this case, the designer has chosen to use a hydraulic connection between the ponding zone and the stone base. Therefore, the ponding zone does not need to temporarily store 75% of the water quality volume.

Step 4: Calculate the volume of voids in the soil media layer assuming a soil media depth of 1.5 ft equal to the minimum soil media depth of 1.5 ft for a stormwater planter practice.

$$V_S = A_{SMP} * D_S * n_S$$

V_S = volume of voids in the soil media layer (cf)
A_{SMP} = area of the SMP (sf) = 300 sf
D_S = depth of soil media layer (ft) = 1.5 ft
n_S = available porosity of soil media (cf/cf) = 0.2 cf/cf

$$V_S = 300\ sf * 1.5\ ft\ * 0.2\frac{cf}{cf}$$

$$V_S = 90\ cf$$

Step 5: Calculate the volume of voids created by internal structures.

Assume there are no internal structures in this stormwater planter practice, so the volume is 0.

$V_I = 0\ cf$

Step 6: Calculate the volume of voids in the drainage layer assuming a drainage media depth of 1 ft, which is equal to the minimum drainage media depth of 1 ft for a stormwater planter practice.

$$V_D = (A_{SMP} * D_D - V_{I,d}) * n_D$$

where:
V_D = volume of voids in the drainage layer (cf)
A_{SMP} = area of the SMP (sf) = 300 cf
D_D = depth of the drainage layer (ft) = 1 ft
$V_{I,d}$ = volume of voids created by internal structures within the drainage layer (cf) = 0 cf
n_D = porosity of drainage layer media (cf/cf) = 0.4 cf/cf

$$V_D = (300\ sf * 1\ ft - 0\ cf) * 0.4 \frac{cf}{cf}$$

$$V_D = 120\ cf$$

Step 7: Calculate the total SMP volume from the individual component volumes and compare to the WQ_V.

$$V_{SMP} = V_P + V_S + V_I + V_D$$

where:
V_{SMP} = storage volume of SMP (cf)
V_P = volume of surface ponding (cf) = 150 cf
V_S = volume of voids in the soil media layer (cf) = 90 cf
V_I = volume of voids created by internal structures such as chambers or pipes (cf) = 0 cf
V_D = volume of voids in the drainage layer (cf) = 120 cf

$$V_{SMP} = 150\ cf + 90\ cf + 0\ cf + 120\ cf$$

$$V_{SMP} = 360\ cf > WQ_V = 356.25\ cf \qquad OK$$

Step 8: Check the ponding and infiltration drawdown times of the practice do not exceed the required times of 12 hours and 48 hours, respectively.

Infiltration drawdown time:

$$dt_{SMP} = \frac{V_{SMP}}{\left(\frac{i}{12}\right) * A_{SMP}}$$

where:

dt_{SMP} = drawdown time of infiltration SMP (hr)

V_{SMP} = volume of infiltration SMP (cf) = WQ_V = 360 cf

i = field measured infiltration rate (in/hr) = 2 in/hr

A_{SMP} = area at the base of infiltration SMP (sf) = 300 sf

$$dt_{SMP} = \frac{360\ cf}{\left(\frac{2\ in/hr}{12}\right) * 300\ sf}$$

$$dt_{SMP} = 7.2\ hr < 48\ hr \qquad OK$$

Surface ponding drawdown time:

$$dt_P = \frac{V_P}{\left(\frac{K_S}{12}\right) * \left(1 + \frac{0.5D_p}{D_m}\right) * \left(\frac{A_{P1} + A_{P2}}{2}\right)}$$

where:

dt_P = drawdown time of surface ponding (hr)

V_P = volume of surface ponding (cf) = 75 cf

K_S = saturated hydraulic conductivity of media below the surface ponding area (in/hr) = 2 in/hr

D_p = maximum depth of ponding (ft) = 0.5 ft

D_m = depth of media below surface ponding area (ft) = 1.5 ft

A_{P1} = area at the base of surface ponding zone (sf) = 300 sf

A_{P2} = area at the top of surface ponding zone (sf) = 300 sf

$$dt_P = \frac{150\ cf}{\left(\frac{2\frac{in}{hr}}{12}\right) * \left(1 + \frac{0.5 * 0.5\ ft}{1.5\ ft}\right) * \left(\frac{300\ sf + 300\ sf}{2}\right)}$$

$$dt_P = 2.57\ hr < 12\ hr \qquad OK$$

Note: A portion of the SMP volume for this practice may be applied towards meeting the V_V requirements, see Chapter 4 and Appendix C.

Evapotranspiration

Green Roof

Design a green roof that will treat the water quality volume from a 1,100 square foot rooftop with a runoff coefficient of 0.95. Assume that the green roof will cover 900 square feet (82%) of the rooftop due to required setbacks and/or equipment.

Step 1: Calculate the WQ_V.

$$WQ_V = \frac{1.5\ in}{12} * A * R_V$$

where:
WQ_V = water quality volume (cf)
A = contributing area (sf) = 1,100 sf
R_V = runoff coefficient relating total rainfall and runoff
R_V = 0.05 + 0.009(I) = 0.95
I = percent impervious cover = 100%

$$WQ_V = \frac{1.5\ in}{12} * 1{,}100\ sf * 0.95$$

$$WQ_V = 130.63\ cf$$

Note: Since the green roof will cover 900 square feet (82% of the total area) and the maximum loading ratio 1:1, the green roof may only treat up to 106.88 cf (82%) of the 130.63 cf water quality volume.

Step 2: Calculate the volume of surface ponding.

Green roofs are fast draining and typically do not pond water. Any ponding that does occur would not be stored long enough for evapotranspiration. Therefore, the volume of surface ponding is zero.

$$V_P = 0\ cf$$

Step 3: Calculate the volume of voids in the soil media layer assuming a soil media depth of 0.33 ft, which is equal to the minimum soil media depth of 0.33 ft for a green roof.

$$V_S = A_{SMP} * D_S * n_S$$

V_S = volume of voids in the soil media layer (cf)
A_{SMP} = area of the SMP (sf) = 900 sf
D_S = depth of soil media layer (ft) = 0.33 ft

n_S = available porosity of soil media (cf/cf) = 0.2 cf/cf

$$V_S = 900\ sf * 0.33\ ft\ * 0.2\frac{cf}{cf}$$

$$V_S = 59.4\ cf$$

Step 4: Calculate the volume of voids created by internal structures.

Assume there are no internal structures in this green roof practice, so the volume is 0.

$$V_I = 0\ cf$$

Step 5: Calculate the volume of voids in the drainage layer.

The active storage zone for a green roof is considered from the base of the soil media up, so the storage volume of the drainage layer is zero.

$$V_D = 0\ cf$$

Step 6: Calculate the total SMP volume from the individual component volumes and compare to the WQ_V.

$$V_{SMP} = V_P + V_S + V_I + V_D$$

where:
V_{SMP} = storage volume of SMP (cf)
V_P = volume of surface ponding (cf) = 0 cf
V_S = volume of voids in the soil media layer (cf) = 59.4 cf
V_I = volume of voids created by internal structures such as chambers or pipes (cf) = 0 cf
V_D = volume of voids in the drainage layer (cf) = 0 cf

$$V_{SMP} = 0\ cf + 59.4\ cf + 0\ cf + 0\ cf$$

$$V_{SMP} = 59.4\ cf < WQ_V = 130.63\ cf \qquad \boldsymbol{NOT\ MET}$$

Since the SMP volume is less than the WQ_V, other practices must be used to treat the remaining WQ_V.

Infiltration (unvegetated)

Subsurface Gallery

Design a subsurface gallery that will treat the water quality volume from an impervious area of 90,000 square feet (2.07 acres) with a runoff coefficient of 0.95. Assume an infiltration rate of 1 in/hr.

Step 1: Calculate the WQ_V.

$$WQ_V = \frac{1.5\ in}{12} * A * \ R_V$$

where:
WQ_V = water quality volume (cf)
A = contributing area (sf) = 90,000 sf
R_V = runoff coefficient relating total rainfall and runoff
R_V = 0.05 + 0.009(I) = 0.95
I = percent impervious cover = 100%

$$WQ_V = \frac{1.5\ in}{12} * 90{,}000\ sf * \ 0.95$$

$$WQ_V = 10{,}687.5\ cf$$

Step 2: Calculate the SMP area assuming a loading ratio of 1:10. Note that the subsurface gallery does not have a maximum loading ratio. Use the area to set the initial length and width of the practice.

$$A_{SMP} = \frac{A}{10}$$

where:
A_{SMP} = area at the base of infiltration SMP (sf)
A = contributing area (sf) = 90,000 sf

$$A_{SMP} = \frac{90{,}000\ sf}{10}$$

$$A_{SMP} = 9{,}000\ sf$$

Assume a 90 ft x 100 ft practice.

Step 3: Calculate the volume of surface ponding.

There is no surface ponding associated with a subsurface gallery since the SMP is below ground level, so the volume is 0.

$$V_P = 0$$

Step 4: Calculate the volume of voids in the soil media layer.

There is no soil media associated with a subsurface gallery, so the volume is 0.

$$V_S = 0$$

Step 5: Calculate the volume of voids created by internal structures.

Assume 300 ft of 12” distribution pipe will be placed within the system in a grid pattern.

$$V_I = A_P * L_P$$

where:
V_I = volume of voids created by internal structure (cf)
A_P = area of pipe (sf) = (π) * $(0.5)^2$ = 0.79 sf
L_P = total length of pipe (ft) = 300 ft

$$V_I = 0.79\ sf * 300\ ft$$

$$V_I = 237\ cf$$

Step 6: Calculate the volume of voids in the drainage layer assuming a drainage media depth of 3 ft, which is greater than the minimum drainage media depth of 1 ft for a subsurface gallery practice.

$$V_D = (A_{SMP} * D_D - V_{I,d}) * n_D$$

where:
V_D = volume of voids in the drainage layer (cf)
A_{SMP} = area of the SMP (sf) = 9,000 sf
D_D = depth of the drainage layer (ft) = 2 ft
$V_{I,d}$ = volume of voids created by internal structures within the drainage layer (cf) = 273 cf
n_D = porosity of drainage layer media (cf/cf) = 0.4 cf/cf

$$V_D = (9{,}000\ sf * 3\ ft - 273\ cf) * 0.4 \frac{cf}{cf}$$

$V_D = 10{,}690.8\ cf$

Step 7: Calculate the total SMP volume from the individual component volumes and compare to the WQ_V.

$V_{SMP} = V_P + V_S + V_I + V_D$

where:
V_{SMP} = storage volume of SMP (cf)
V_P = volume of surface ponding (cf) = 0 cf
V_S = volume of voids in the soil media layer (cf) = 0 cf
V_I = volume of voids created by internal structures such as chambers or pipes (cf) = 273 cf
V_D = volume of voids in the drainage layer (cf) = 10,690.8 cf

$V_{SMP} = 0\ cf + 0\ cf + 273\ cf + 10{,}690.8\ cf$

$V_{SMP} = 10{,}963.8\ cf > WQ_V = 10{,}687.5\ cf \qquad OK$

Step 8: Check the infiltration drawdown time does not exceed the required time of 48 hours.

$$dt_{SMP} = \frac{V_{SMP}}{\left(\frac{i}{12}\right) * A_{SMP}}$$

where:
dt_{SMP} = drawdown time of infiltration SMP (hr)
V_{SMP} = volume of infiltration SMP (cf) = WQ_V = 10,963.8 cf
i = field measured infiltration rate (in/hr) = 1 in/hr
A_{SMP} = area at the base of infiltration SMP (sf) = 9,000 sf

$$dt_{SMP} = \frac{10{,}963.8\ cf}{\left(\frac{1\ in/hr}{12}\right) * 9{,}000\ sf}$$

$dt_{SMP} = 14.62\ hr < 48\ hr \qquad OK$

Note: A portion of the SMP volume for this practice may be applied towards meeting the V_V requirements, see Chapter 4 and Appendix C.

Reuse

Cistern

Design a reuse system to treat the water quality volume from a 3,000 square foot impervious surface with a runoff coefficient of 0.95. Designers must additionally show that water will be reused for non-irrigation purposes.

Step 1: Calculate the WQ_V.

$$WQ_V = \frac{1.5\ in}{12} * A * \ R_V$$

where:
WQ_V = water quality volume (cf)
A = contributing area (sf) = 3,000 sf
R_V = runoff coefficient relating total rainfall and runoff
R_V = 0.05 + 0.009(I) = 0.95
I = percent impervious cover = 100%

$$WQ_V = \frac{1.5\ in}{12} * 3{,}000\ sf * \ 0.95$$

$$WQ_V = 356.25\ cf$$

Step 2: Calculate the total SMP volume from unit conversion of the WQ_V.

$$V_{SMP} = WQ_V * (7.5\,\frac{gal}{cf})$$

$$V_{SMP} = 356.25\ cf * (7.5\,\frac{gal}{cf})$$

$$V_{SMP} = 2{,}671.88\ gal$$

Therefore, to treat the water quality volume for the area draining to the practice, a 2,700-gallon cistern is required.

Note: The system may be designed larger if more water is needed for the intended reuse application.

Filtration

Bioretention

Design a bioretention practice that will treat the water quality volume from an impervious area of 21,780 square feet (0.5 acres), with a runoff coefficient of 0.95. Note that filtration system may only be used to treat the water quality volume in separate storm sewer areas. Assume a soil media saturated hydraulic conductivity of 2 in/hr.

Step 1: Calculate the WQ_V.

$$WQ_V = \frac{1.5\ in}{12} * A * \ R_V$$

where:
WQ_V = water quality volume (cf)
A = contributing area (sf) = 21,780 sf
R_V = runoff coefficient relating total rainfall and runoff
R_V = 0.05 + 0.009(I) = 0.95
I = percent impervious cover = 100%

$$WQ_V = \frac{1.5\ in}{12} * 21{,}780\ sf * \ 0.95$$

$$WQ_V = 2{,}586.38\ cf$$

Step 2: Calculate the SMP area assuming a loading ratio of 1:8, which is less than the maximum loading ratio of 1:20 for a bioretention practice. Use the area to set the initial length and width of the practice.

$$A_{SMP} = \frac{A}{8}$$

where:
A_{SMP} = area at the base of infiltration SMP (sf)
A = contributing area (sf) = 21,780 sf

$$A_{SMP} = \frac{21{,}780\ sf}{8}$$

$$A_{SMP} = 2{,}722.5\ sf$$

Round the SMP area up to 2,730 sf. Assume a 65 ft x 42 ft practice.

Step 3: Calculate the volume of surface ponding assuming the maximum surface ponding depth of 1 ft for a bioretention practice.

Assume the ponding zone is uniformly sloped. Use the SMP area and grading of the practice to determine the area at the base and top of the surface ponding zone.

$$V_P = \frac{1}{3}\left(A_{P1} + \sqrt{A_{P1} * A_{P2}} + A_{P2}\right) * D_P$$

where:
V_P = volume of surface ponding (cf)
A_{P1} = area at the base of surface ponding zone (sf) = 1,400 sf
A_{P2} = area at the top of surface ponding zone (sf) = 2,600 sf
D_P = depth of ponding (ft) = 1 ft

$$V_P = \frac{1}{3}\left(1{,}400\ sf + \sqrt{1{,}400\ sf * 2{,}600\ sf} + 2{,}600\ sf\right) * 1\ ft$$

$$V_P = 1{,}969.29\ cf$$

Since a hydraulic connection is not being used, confirm that the volume of surface ponding is greater than 75% of the water quality volume.

$$V_P = 1{,}969.29\ cf < 75\%\ of\ WQ_V = 1{,}939.79\ cf \qquad OK$$

Step 4: Calculate the volume of voids in the soil media layer assuming a soil media depth of 3.5 ft, which is greater than the minimum soil media depth of 2.5 ft for bioretention practices.

$$V_S = A_{SMP} * D_S * n_S$$

V_S = volume of voids in the soil media layer (cf)
A_{SMP} = area of the SMP (sf) = 2,730 sf
D_S = depth of soil media layer (ft) = 3.5 ft
n_S = available porosity of soil media (cf/cf) = 0.2 cf/cf

$$V_S = 2{,}730\ sf * 3.5\ ft\ * 0.2\frac{cf}{cf}$$

$$V_S = 1{,}911\ cf$$

Step 5: Calculate the volume of voids created by internal structures.

Assume 92 ft of 12” distribution pipe will be placed within the system in a grid pattern.

$V_I = A_P * L_P$

where:
V_I = volume of voids created by internal structure (cf)
A_P = area of pipe (sf) = (π) * $(0.5)^2$ = 0.79 sf
L_P = total length of pipe (ft) = 92 ft

$V_I = 0.79\ sf * 92\ sf$

$V_I = 72.68\ cf$

Step 6: Calculate the volume of voids in the drainage layer assuming a drainage media depth of 3 ft, which is greater than the minimum drainage media depth of 1 ft for bioretention practices.

$V_D = (A_{SMP} * D_D - V_{I,d}) * n_D$

where:
V_D = volume of voids in the drainage layer (cf)
A_{SMP} = area of the SMP (sf) = 2,730 cf
D_D = depth of the drainage layer (ft) = 3 ft
$V_{I,d}$ = volume of voids created by internal structures within the drainage layer (cf) = 72.68 cf
n_D = porosity of drainage layer media (cf/cf) = 0.4 cf/cf

$V_D = (2{,}730\ sf * 3\ ft - 72.68\ cf) * 0.4 \frac{ft^3}{ft^3}$

$V_D = 3{,}246.93\ cf$

Step 7: Calculate the total SMP volume from the individual component volumes and compare to the WQ_V.

$V_{SMP} = V_P + V_S + V_I + V_D$

where:
V_{SMP} = storage volume of SMP (cf)
V_P = volume of surface ponding (cf) = 1,969.29 cf
V_S = volume of voids in the soil media layer (cf) = 1,911 cf
V_I = volume of voids created by internal structures such as chambers or pipes (cf) = 72.68 cf
V_D = volume of voids in the drainage layer (cf) = 3,246.93 cf

$V_{SMP} = 1{,}969.29\ cf + 1{,}911\ cf + 72.68\ cf + 3{,}246.93\ cf$

$V_{SMP} = 7{,}199.9\ cf > WQ_V = 2{,}586.38\ cf \qquad OK$

Step 8: Check the ponding and filtration drawdown times of the practice do not exceed the required times of 24 hours and 48 hours, respectively.

Filtration drawdown time:

$$dt_{SMP} = \frac{V_{SMP}}{\left(\frac{K_S}{12}\right) * \left(1 + \frac{0.5D_{pf}}{D_f}\right) * A_f}$$

where:
dt_{SMP} = drawdown time of filtration SMP (hr)
V_{SMP} = volume of filtration SMP (cf) = 7,199.9 cf
K_S = saturated hydraulic conductivity of filter media (in/hr) = 2 in/hr
D_{pf} = maximum depth of ponding above filter media (ft) = 1 ft
D_f = depth of filter media (ft) = 3.5 ft
A_f = area of filter bed (sf) = 2,730 sf

$$dt_{SMP} = \frac{7{,}199.9\ cf}{\left(\frac{2\frac{in}{hr}}{12}\right) * \left(1 + \frac{0.5 * 1\ ft}{3.5\ ft}\right) * 2{,}730\ sf}$$

$$dt_{SMP} = 13.85\ hr < 48\ hr \qquad OK$$

Surface ponding drawdown time:

$$dt_P = \frac{V_P}{\left(\frac{K_S}{12}\right) * \left(1 + \frac{0.5D_p}{D_m}\right) * \left(\frac{A_{P1} + A_{P2}}{2}\right)}$$

where:
dt_P = drawdown time of surface ponding (hr)
V_P = volume of surface ponding (cf) = 1,969.29 cf
K_S = saturated hydraulic conductivity of media below the surface ponding area (in/hr) = 2 in/hr
D_p = maximum depth of ponding (ft) = 1 ft
D_m = depth of media below surface ponding area (ft) = 3.5 ft
A_{P1} = area at the base of surface ponding zone (sf) = 1,400 sf
A_{P2} = area at the top of surface ponding zone (sf) = 2,600 sf

$$dt_P = \frac{1{,}969.29\ cf}{\left(\frac{2\frac{in}{hr}}{12}\right) * \left(1 + \frac{0.5 * 1\ ft}{3.5\ ft}\right) * \left(\frac{1{,}400\ sf + 2{,}600\ sf}{2}\right)}$$

$$dt_P = 5.17\ hr < 24\ hr \qquad OK$$

SEWER OPERATIONS VOLUME SIZING EXAMPLES

Detention

Detention Tank - CSS with SCP

A 93,200 sf site in the Bronx consists of a multistory commercial building. The site was proposed to connect to a 15 in. combined sewer. Design a detention tank to treat the sewer operations volume (V_V), given the following:

Area = 93,200 sf
Roof = 29,000 sf @ 0.95 runoff coefficient
Paved = 48,000 sf @ 0.85 runoff coefficient
Grass = 16,200 sf @ 0.20 runoff coefficient

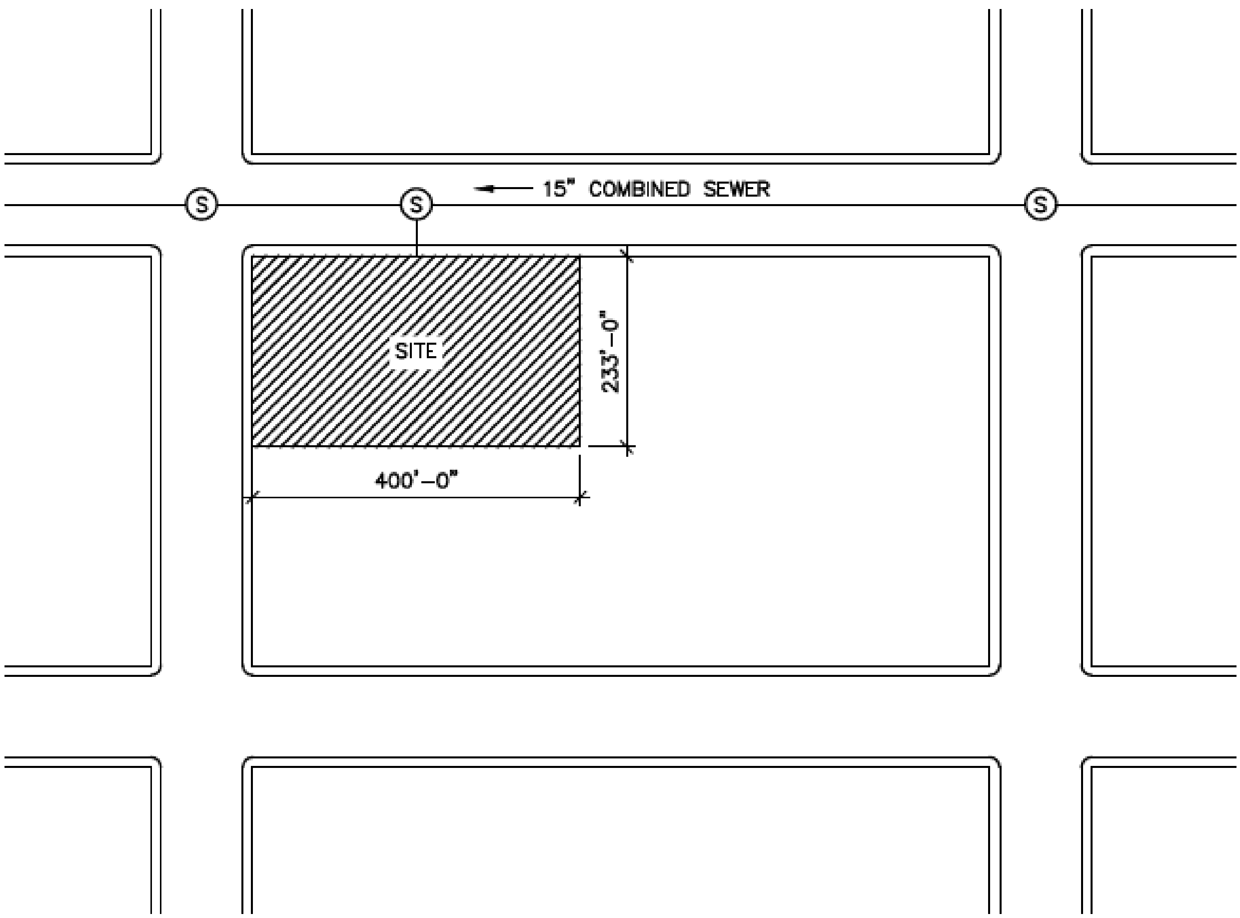


Figure F.1. Schematic of Site (Not to Scale)

Step 1: Identify the rainfall depth (R_D) based on the sewershed type and connection proposal type for the project. Use Table 2.7 in Chapter 2.

Since the project is 20,000 sf or more, and consists of a multistory commercial building, this project requires a site connection permit (SCP). In addition, the site is connecting to a 15 in. combined sewer.

Table 2.7. Applied rainfall depth by sewershed type and connection proposal type.

R_D	Description
1.85	CSS areas with SCP
1.50	CSS areas with HCP
1.50	MS4 areas wiith SCP
1.10	MS4 areas with HCP

According to Table 2.7, $R_D = 1.85\ in.$

Step 2: Calculate the runoff coefficient (C_W) using the weighted area approach.

$$C_W = \frac{(C_1A_1 + C_2A_2 + \cdots etc.)}{A_t}$$

where:
C_W = weighted runoff coefficient relating peak rate of rainfall and runoff
C_1 = the runoff coefficient for the area classified as roof = 0.95
A_1 = the area classified as roof (sf) = 29,000 sf
C_2 = the runoff coefficient for the area classified as paved = 0.85
A_2 = the area classified as paved (sf) = 48,000 sf
C_3 = the runoff coefficient for the area classified as grass = 0.20
A_3 = the area classified as grass (sf) = 16,200 sf
A_t = contributing area (sf) = 93,200 sf

$$C_W = \frac{(0.95 * 29{,}000\ sf) + (0.85 * 48{,}000\ sf) + (0.20 * 16{,}200\ sf)}{93{,}200\ sf}$$

$$C_W = 0.768$$

Step 3: Calculate V_V.

$$V_V = \frac{R_D}{12} * A * C_W$$

where:
V_V = sewer operations volume (cf)
R_D = rainfall depth (in) = 1.85 in
A = contributing area (sf) = 93,200 sf
C_W = weighted runoff coefficient relating peak rate of rainfall and runoff = 0.768

$$V_V = \frac{1.85\ in}{12} * 93{,}200\ sf * 0.768$$

$$V_V = 11{,}035\ cf$$

Step 4: Calculate the release rate to be maintained by the controlled-flow orifice. Use the maximum release rate per acre (q) shown in Table 2.9, Chapter 2.

The site is connecting to a 15 in. combined sewer.

Table 2.9. Maximum release rate per acre (cfs/acre) by sewershed type.

q (cfs/acre)	Description
1.0	MS4 areas
0.1	CSS areas

According to Table 2.9, $q = 0.1\,\frac{cfs}{acre}$.

$$Q_{DRR} = \frac{q\ * A}{43560}\ or\ 0.046\ \ [whichever\ is\ greater]$$

where:
Q_{DRR} = maximum release rate for the site (cfs)
q = maximum release rate per acre (cfs/acre) = 0.1 cfs/acre
A = contributing area (sf) = 93,200 sf

$$Q_{DRR} = \frac{0.1\ \frac{cfs}{acre}\ * 93{,}200\ sf}{43560}\ or\ 0.046\ \ [whichever\ is\ greater]$$

$$Q_{DRR} = 0.214\ cfs\ > 0.046\ cfs$$

The maximum release rate is 0.214 cfs.

Step 5: Use the controlled-flow orifice equation to determine an appropriate orifice area by assuming the active storage depth.

In order to minimize the area required for the detention tank, choose the maximum depth that is still feasible according to site limitations and use a re-entrant orifice. In this case, the designer has chosen an active storage depth of 4 ft.

$$Q_O = C_D * A_o * \sqrt{2gH}$$

where:
Q_O = maximum release rate of orifice (cfs) = 0.214 cfs
C_D = coefficient of discharge, 0.52 for re-entrant orifice
A_O = area of orifice (sf)
g = acceleration due to gravity, 32.2 (ft/s^2)

H = maximum hydraulic head above the centerline of the orifice (ft) = 4 ft

$$0.214\ cfs = 0.52 * A_o * \sqrt{2 * 32.2\ \left(\frac{ft}{s^2}\right) * 4\ ft}$$

$$A_o = 0.026\ sf$$

Step 6: Translate the area of the controlled-flow orifice (A_O) into a diameter and check that it is greater than the minimum diameter of 1 in.

$$A_o = \frac{\left[\pi * \left(\frac{D_o}{2}\right)^2\right]}{144}$$

where:
A_O = area of orifice (sf) = 0.026 sf
D_O = diameter of orifice (in)

$$0.026\ sf = \frac{\left[\pi * \left(\frac{D_O}{2}\right)^2\right]}{144}$$

$$D_O = 2.18\ in\ > 1\ in \qquad OK$$

Set the orifice diameter to the nearest 0.25-inch interval rounding down, with a minimum orifice diameter of one-inch. In this case, use an orifice diameter of 2.00 inches.

Step 7: Confirm the orifice area of the selected orifice diameter from Step 6.

$$A_o = \frac{\left[\pi * \left(\frac{D_o}{2}\right)^2\right]}{144}$$

where:
A_O = area of orifice (sf)
D_O = diameter of orifice (in) = 2 in

$$A_O = \frac{\left[\pi * \left(\frac{2\ in}{2}\right)^2\right]}{144}$$

$$A_O = 0.022\ sf$$

Step 8: Confirm the required active storage depth in the tank using the orifice area from Step 7.

$$Q_O = C_D * A_o * \sqrt{2gH}$$

where:

Q_O = maximum release rate of orifice (cfs) = 0.214 cfs

C_D = coefficient of discharge, 0.52 for re-entrant orifice

A_O = area of orifice (sf) = 0.022 sf

g = acceleration due to gravity, 32.2 (ft/s^2)

H = maximum hydraulic head above the centerline of the orifice (ft)

$$0.214\ cfs = 0.52 * 0.022\ sf * \sqrt{2 * 32.2\ \left(\frac{ft}{s^2}\right) * H}$$

$$H = 5.4\ ft$$

If the active storage depth is too high, then increase the orifice size by 0.25 inches and re-run Steps 7-8 until a suitable depth is identified. If the active storage depth is too low, then decrease the orifice size by 0.25 inches (but not less than 1 inch) and re-run Steps 7-8. Alternatively, the designer can choose a different orifice configuration as needed to modify the active storage depth.

In this case, the depth is feasible.

Step 9: Set the dimensions of the detention tank's active storage zone.

Based on the active storage depth of 5.4 ft and the V_V of 11,035 cf, set the interior detention tank dimensions to L: 45.5 ft and W: 45.5 ft. The resulting detention tank has an active storage volume of 11,179 cf. Note that the exterior dimensions of the detention tank will be larger than the dimensions of the active storage zone (45.5'L x 45.5'W x 5.4'D) to accommodate wall thickness, bypass structures, and/or other internal features.

Detention Tank - CSS with HCP

A 15,000 sf site in the Bronx consists of a two-family (no-fee) residence. The site was proposed to connect to a 15 in. combined sewer. Design a detention tank to treat the sewer operations volume (V_V), given the following:

Area = 15,000 sf
Roof = 2,000 sf @ 0.95 runoff coefficient
Paved = 7,000 sf @ 0.85 runoff coefficient
Grass = 6,000 sf @ 0.20 runoff coefficient

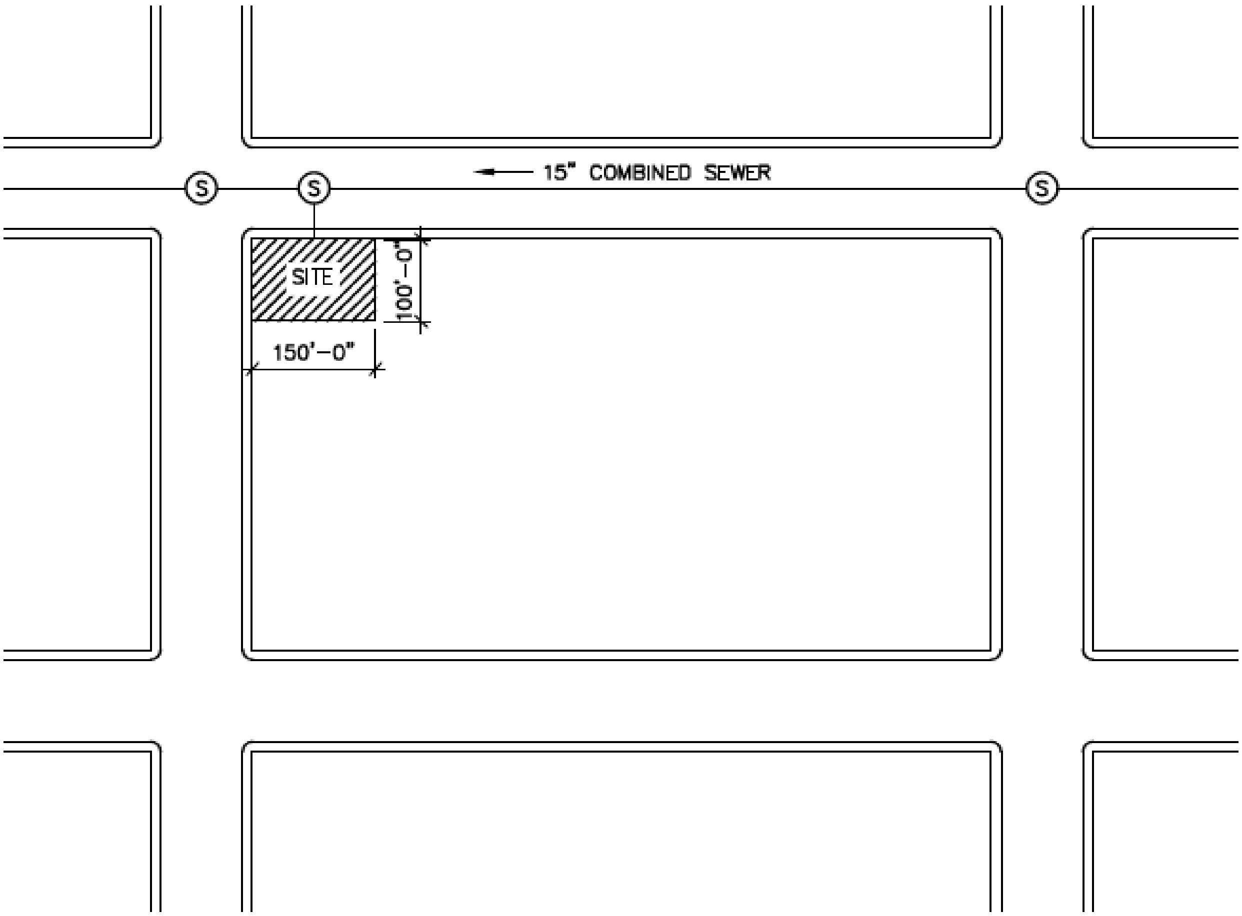


Figure F.2. Schematic of Site (Not to Scale)

Step 1: Identify the rainfall depth (R_D) based on the sewershed type and connection proposal type for the project. Use Table 2.7 in Chapter 2.

Since the project is less than 20,000 sf and consists of a two-family (no fee) residence, this project requires a house connection permit (HCP). In addition, the site is connecting to a 15 in. combined sewer.

Table 2.7. Applied rainfall depth by sewershed type and connection proposal type.

R_D	Description
1.85	CSS areas with SCP
1.50	CSS areas with HCP
1.50	MS4 areas wiith SCP
1.10	MS4 areas with HCP

According to Table 2.7, $R_D = 1.50\ in.$

Step 2: Calculate the runoff coefficient (C_W) using the weighted area approach.

$$C_W = \frac{(C_1 A_1 + C_2 A_2 + \cdots etc.)}{A_t}$$

where:
C_W = weighted runoff coefficient relating peak rate of rainfall and runoff
C_1 = the runoff coefficient for the area classified as roof = 0.95
A_1 = the area classified as roof (sf) = 2,000 sf
C_2 = the runoff coefficient for the area classified as paved = 0.85
A_2 = the area classified as paved (sf) = 7,000 sf
C_3 = the runoff coefficient for the area classified as grass = 0.20
A_3 = the area classified as grass (sf) = 6,000 sf
A_t = contributing area (sf) = 15,000 sf

$$C_W = \frac{(0.95 * 2{,}000\ sf) + (0.85 * 7{,}000\ sf) + (0.20 * 6{,}000\ sf)}{15{,}000\ sf}$$

$$C_W = 0.603$$

Step 3: Calculate V_V.

$$V_V = \frac{R_D}{12} * A * C_W$$

where:
V_V = sewer operations volume (cf)
R_D = rainfall depth (in) = 1.50 in
A = contributing area (sf) = 15,000 sf
C_W = weighted runoff coefficient relating peak rate of rainfall and runoff = 0.603

$$V_V = \frac{1.50\ in}{12} * 15{,}000\ sf * 0.603$$

$V_V = 1{,}131\ cf$

Step 4: Calculate the release rate to be maintained by the controlled-flow orifice. Use the maximum release rate per acre (q) shown in Table 2.9, Chapter 2.

The site is connecting to a 15 in. combined sewer.

Table 2.9. Maximum release rate per acre (cfs/acre) by sewershed type.

q (cfs/acre)	Description
1.0	MS4 areas
0.1	CSS areas

According to Table 2.9, $q = 0.1 \frac{cfs}{acre}$.

$$Q_{DRR} = \frac{q\ * A}{43560}\ or\ 0.046\ \ [whichever\ is\ greater]$$

where:

Q_{DRR} = maximum release rate for the site (cfs)

q = maximum release rate per acre (cfs/acre) = 0.1 cfs/acre

A = contributing area (sf) = 15,000 sf

$$Q_{DRR} = \frac{0.1\ \frac{cfs}{acre} * 15{,}000\ sf}{43560}\ or\ 0.046\ \ [whichever\ is\ greater]$$

$$Q_{DRR} = 0.034\ cfs\ < 0.046\ cfs$$

The maximum release rate is 0.046 cfs.

Step 5: Use the controlled-flow orifice equation to determine an appropriate orifice area by assuming the active storage depth.

In order to minimize the area required for the detention tank, choose the maximum depth that is still feasible according to site limitations and use a re-entrant orifice. In this case, the designer has chosen an active storage depth of 4 ft.

$$Q_O = C_D * A_o * \sqrt{2gH}$$

where:

Q_O = maximum release rate of orifice (cfs) = 0.046 cfs

C_D = coefficient of discharge, 0.52 for re-entrant orifice

A_O = area of orifice (sf)

g = acceleration due to gravity, 32.2 (ft/s^2)

H = maximum hydraulic head above the centerline of the orifice (ft) = 4 ft

$$0.046\ cfs = 0.52 * A_o * \sqrt{2 * 32.2\ \left(\frac{ft}{s^2}\right) * 4\ ft}$$

$$A_o = 0.006\ sf$$

Step 6: Translate the area of the controlled-flow orifice (A_O) into a diameter and check that it is greater than the minimum diameter of 1 in.

$$A_o = \frac{\left[\pi * \left(\frac{D_o}{2}\right)^2\right]}{144}$$

where:
A_O = area of orifice (sf) = 0.006 sf
D_O = diameter of orifice (in)

$$0.006\ sf = \frac{\left[\pi * \left(\frac{D_O}{2}\right)^2\right]}{144}$$

$$D_O = 1.05\ in\ > 1\ in \qquad OK$$

Set the orifice diameter to the nearest 0.25-inch interval rounding down, with a minimum orifice diameter of one-inch. In this case, use an orifice diameter of 1.00 inch.

Step 7: Confirm the orifice area of the selected orifice diameter from Step 6.

$$A_o = \frac{\left[\pi * \left(\frac{D_o}{2}\right)^2\right]}{144}$$

where:
A_O = area of orifice (sf)
D_O = diameter of orifice (in) = 1 in

$$A_O = \frac{\left[\pi * \left(\frac{1\ in}{2}\right)^2\right]}{144}$$

$$A_O = 0.005\ sf$$

Step 8: Confirm the required active storage depth in the tank using the orifice area from Step 7.

$$Q_O = C_D * A_o * \sqrt{2gH}$$

where:
Q_O = maximum release rate of orifice (cfs) = 0.046 cfs
C_D = coefficient of discharge, 0.52 for re-entrant orifice
A_O = area of orifice (sf) = 0.005 sf
g = acceleration due to gravity, 32.2 (ft/s^2)
H = maximum hydraulic head above the centerline of the orifice (ft)

$$0.046\ cfs = 0.52 * 0.005\ sf * \sqrt{2 * 32.2\ \left(\frac{ft}{s^2}\right) * H}$$

$$H = 4.9\ ft$$

If the active storage depth is too high, then increase the orifice size by 0.25 inches and re-run Steps 7-8 until a suitable depth is identified. If the active storage depth is too low, then decrease the orifice size by 0.25 inches (but not less than 1 inch) and re-run Steps 7-8. Alternatively, the designer can choose a different orifice configuration as needed to modify the active storage depth.

In this case, the depth is feasible.

Step 9: Set the dimensions of the detention tank's active storage zone.

Based on the active storage depth of 4.9 ft and the V_V of 1,131 cf, set the interior detention tank dimensions to L: 15.5 ft and W: 15.5 ft. The resulting detention tank has an active storage volume of 1,177 cf. Note that the exterior dimensions of the detention tank will be larger than the dimensions of the active storage zone (15.5'L x 15.5'W x 4.9'D) to accommodate wall thickness, bypass structures, and/or other internal features.

Detention Tank - MS4 with SCP

A 25,050 sf site consists of a multistory commercial building. The site was proposed to connect to a 12 in. storm sewer that eventually discharges into Gravesend Bay via an MS4 outfall. Design a detention tank to treat the sewer operations volume (V_V), given the following:

Area = 25,050 sf
Roof = 16,000 sf @ 0.95 runoff coefficient
Paved = 6,100 sf @ 0.85 runoff coefficient
Grass = 2,950 sf @ 0.20 runoff coefficient

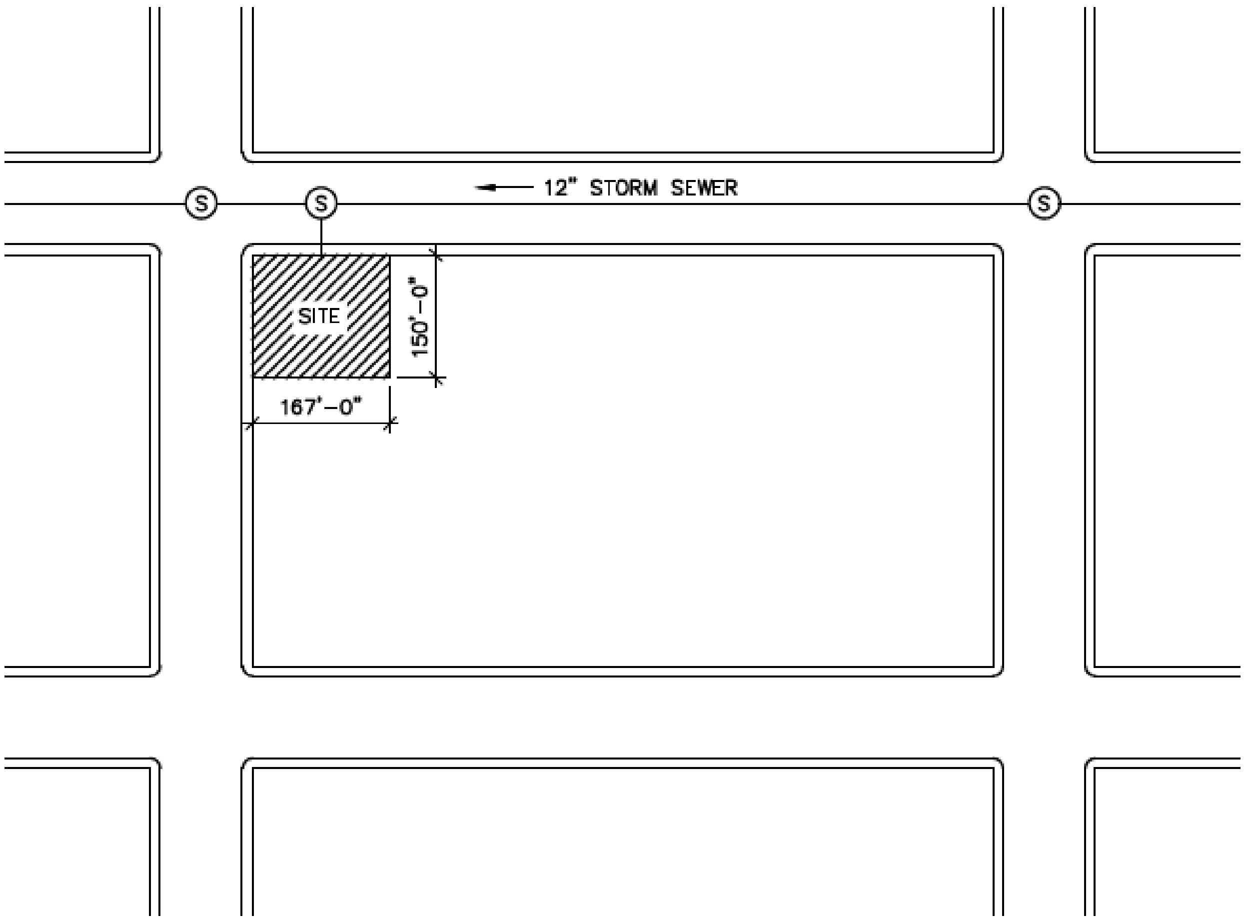


Figure F.3. Schematic of Site (Not to Scale)

Step 1: Identify the rainfall depth (R_D) based on the sewershed type and connection proposal type for the project. Use Table 2.7 in Chapter 2.

Since the project is 20,000 sf or more, and consists of a multistory commercial building, this project requires a site connection permit (SCP). In addition, the site is connecting to a 12 in. storm sewer that discharges through an MS4 outfall.

Table 2.7. Applied rainfall depth by sewershed type and connection proposal type.

R_D	Description
1.85	CSS areas with SCP
1.50	CSS areas with HCP
1.50	MS4 areas wiith SCP
1.10	MS4 areas with HCP

According to Table 2.7, $R_D = 1.50\ in.$

Step 2: Calculate the runoff coefficient (C_W) using the weighted area approach.

$$C_W = \frac{(C_1 A_1 + C_2 A_2 + \cdots etc.)}{A_t}$$

where:
C_W = weighted runoff coefficient relating peak rate of rainfall and runoff
C_1 = the runoff coefficient for the area classified as roof = 0.95
A_1 = the area classified as roof (sf) = 16,000 sf
C_2 = the runoff coefficient for the area classified as paved = 0.85
A_2 = the area classified as paved (sf) = 6,100 sf
C_3 = the runoff coefficient for the area classified as grass = 0.20
A_3 = the area classified as grass (sf) = 2,950 sf
A_t = contributing area (sf) = 25,050 sf

$$C_W = \frac{(0.95 * 16{,}000\ sf) + (0.85 * 6{,}100\ sf) + (0.20 * 2{,}950\ sf)}{25{,}050\ sf}$$

$$C_W = 0.837$$

Step 3: Calculate V_V.

$$V_V = \frac{R_D}{12} * A * C_W$$

where:
V_V = sewer operations volume (cf)
R_D = rainfall depth (in) = 1.50 in
A = contributing area (sf) = 25,050 sf
C_W = weighted runoff coefficient relating peak rate of rainfall and runoff = 0.837

$$V_V = \frac{1.50\ in}{12} * 25{,}050\ sf * 0.837$$

$$V_V = 2{,}621\ cf$$

Step 4: Calculate the release rate to be maintained by the controlled-flow orifice. Use the maximum release rate per acre (q) shown in Table 2.9, Chapter 2.

The site is connecting to a 12 in. storm sewer that discharges through an MS4 outfall.

Table 2.9. Maximum release rate per acre (cfs/acre) by sewershed type.

q (cfs/acre)	Description
1.0	MS4 areas
0.1	CSS areas

According to Table 2.9, $q = 1.0 \frac{cfs}{acre}$.

$$Q_{DRR} = \frac{q\ * A}{43560}\ or\ 0.046\ \ [whichever\ is\ greater]$$

where:

Q_{DRR} = maximum release rate for the site (cfs)

q = maximum release rate per acre (cfs/acre) = 1.0 cfs/acre

A = contributing area (sf) = 25,050 sf

$$Q_{DRR} = \frac{1.0\ \frac{cfs}{acre}\ * 25{,}050\ sf}{43560}\ or\ 0.046\ \ [whichever\ is\ greater]$$

$$Q_{DRR} = 0.575\ cfs\ > 0.046\ cfs$$

The maximum release rate is 0.575 cfs.

Step 5: Use the controlled-flow orifice equation to determine an appropriate orifice area by assuming the active storage depth.

In order to minimize the area required for the detention tank, choose the maximum depth that is still feasible according to site limitations and use a re-entrant orifice. In this case, the designer has chosen an active storage depth of 4 ft.

$$Q_O = C_D * A_o * \sqrt{2gH}$$

where:

Q_O = maximum release rate of orifice (cfs) = 0.575 cfs

C_D = coefficient of discharge, 0.52 for re-entrant orifice

A_O = area of orifice (sf)

g = acceleration due to gravity, 32.2 (ft/s^2)
H = maximum hydraulic head above the centerline of the orifice (ft) = 4 ft

$$0.575\ cfs = 0.52 * A_o * \sqrt{2 * 32.2\ \left(\frac{ft}{s^2}\right) * 4\ ft}$$

$$A_o = 0.069\ sf$$

Step 6: Translate the area of the controlled-flow orifice (A_O) into a diameter and check that it is greater than the minimum diameter of 1 in.

$$A_o = \frac{\left[\pi * \left(\frac{D_o}{2}\right)^2\right]}{144}$$

where:
A_O = area of orifice (sf) = 0.069 sf
D_O = diameter of orifice (in)

$$0.069\ sf = \frac{\left[\pi * \left(\frac{D_O}{2}\right)^2\right]}{144}$$

$$D_O = 3.56\ in\ > 1\ in \qquad OK$$

Set the orifice diameter to the nearest 0.25-inch interval rounding down, with a minimum orifice diameter of one-inch. In this case, use an orifice diameter of 3.50 inches.

Step 7: Confirm the orifice area of the selected orifice diameter from Step 6.

$$A_o = \frac{\left[\pi * \left(\frac{D_o}{2}\right)^2\right]}{144}$$

where:
A_O = area of orifice (sf)
D_O = diameter of orifice (in) = 3.50 inches

$$A_O = \frac{\left[\pi * \left(\frac{3.50\ in}{2}\right)^2\right]}{144}$$

$$A_O = 0.067\ sf$$

Step 8: Confirm the required active storage depth in the tank using the orifice area from Step 7.

$$Q_O = C_D * A_o * \sqrt{2gH}$$

where:

Q_O = maximum release rate of orifice (cfs) = 0.575 cfs

C_D = coefficient of discharge, 0.52 for re-entrant orifice

A_O = area of orifice (sf) = 0.067 sf

g = acceleration due to gravity, 32.2 (ft/s^2)

H = maximum hydraulic head above the centerline of the orifice (ft)

$$0.575\ cfs = 0.52 * 0.067\ sf * \sqrt{2 * 32.2\ \left(\frac{ft}{s^2}\right) * H}$$

$$H = 4.2\ ft$$

If the active storage depth is too high, then increase the orifice size by 0.25 inches and re-run Steps 7-8 until a suitable depth is identified. If the active storage depth is too low, then decrease the orifice size by 0.25 inches (but not less than 1 inch) and re-run Steps 7-8. Alternatively, the designer can choose a different orifice configuration as needed to modify the active storage depth.

In this case, the depth is feasible.

Step 9: Set the dimensions of the detention tank's active storage zone.

Based on the active storage depth of 4.2 ft and the V_V of 2,621 cf, set the interior detention tank dimensions to L: 25 ft and W: 25 ft. The resulting detention tank has an active storage volume of 2,625 cf. Note that the exterior dimensions of the detention tank will be larger than the dimensions of the active storage zone (25'L x 25'W x 4.2'D) to accommodate wall thickness, bypass structures, and/or other internal features.

Detention Tank - MS4 with HCP

A 3,000 sf site consists of a one-family (no-fee) residence. The site was proposed to connect to a 12 in. storm sewer that eventually discharges into East River via an MS4 outfall. Design a detention tank to treat the sewer operations volume (V_V), given the following:

Area = 3,000 sf
Roof = 2,100 sf @ 0.95 runoff coefficient
Paved = 500 sf @ 0.85 runoff coefficient
Grass = 400 sf @ 0.20 runoff coefficient

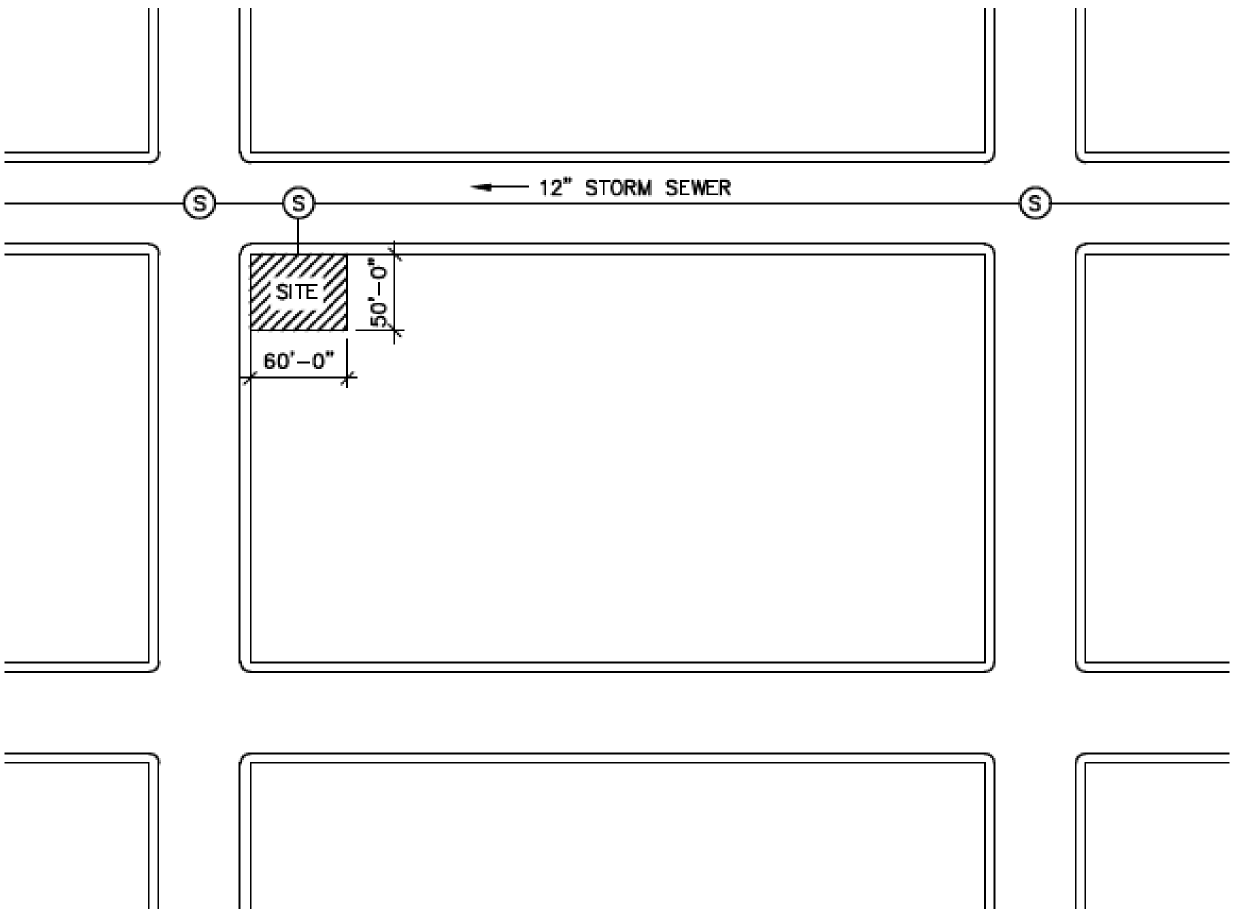


Figure F.4. Schematic of Site (Not to Scale)

Step 1: Identify the rainfall depth (R_D) based on the sewershed type and connection proposal type for the project. Use Table 2.7 in Chapter 2.

Since the project is less than 20,000 sf and consists of a one-family (no fee) residence, this project requires a house connection permit (HCP). In addition, the site is connecting to a 12 in. storm sewer that discharges through an MS4 outfall.

Table 2.7. Applied rainfall depth by sewershed type and connection proposal type.

R_D	Description
1.85	CSS areas with SCP
1.50	CSS areas with HCP
1.50	MS4 areas wiith SCP
1.10	MS4 areas with HCP

According to Table 2.7, $R_D = 1.10\ in.$

Step 2: Calculate the runoff coefficient (C_W) using the weighted area approach.

$$C_W = \frac{(C_1A_1 + C_2A_2 + \cdots etc.)}{A_t}$$

where:

C_W = weighted runoff coefficient relating peak rate of rainfall and runoff

C_1 = the runoff coefficient for the area classified as roof = 0.95

A_1 = the area classified as roof (sf) = 2,100 sf

C_2 = the runoff coefficient for the area classified as paved = 0.85

A_2 = the area classified as paved (sf) = 500 sf

C_3 = the runoff coefficient for the area classified as grass = 0.20

A_3 = the area classified as grass (sf) = 400 sf

A_t = contributing area (sf) = 3,000 sf

$$C_W = \frac{(0.95 * 2{,}100\ sf) + (0.85 * 500\ sf) + (0.20 * 400\ sf)}{3{,}000\ sf}$$

$$C_W = 0.833$$

Step 3: Calculate V_V.

$$V_V = \frac{R_D}{12} * A * C_W$$

where:

V_V = sewer operations volume (cf)

R_D = rainfall depth (in) = 1.10 in

A = contributing area (sf) = 3,000 sf

C_W = weighted runoff coefficient relating peak rate of rainfall and runoff = 0.833

$$V_V = \frac{1.10\ in}{12} * 3{,}000\ sf * 0.833$$

$$V_V = 229\ cf$$

Step 4: Calculate the release rate to be maintained by the controlled-flow orifice. Use the maximum release rate per acre (q) shown in Table 2.9, Chapter 2.

The site is connecting to a 12 in. storm sewer that discharges through an MS4 outfall.

Table 2.9. Maximum release rate per acre (cfs/acre) by sewershed type.

q (cfs/acre)	Description
1.0	MS4 areas
0.1	CSS areas

According to Table 2.9, $q = 1.0\frac{cfs}{acre}$.

$$Q_{DRR} = \frac{q\ * A}{43560}\ or\ 0.046\ \ [whichever\ is\ greater]$$

where:

Q_{DRR} = maximum release rate for the site (cfs)

q = maximum release rate per acre (cfs/acre) = 1.0 cfs/acre

A = contributing area (sf) = 3,000 sf

$$Q_{DRR} = \frac{1.0\ \frac{cfs}{acre}\ * 3{,}000\ sf}{43560} or\ 0.046\ \ [whichever\ is\ greater]$$

$$Q_{DRR} = 0.069\ cfs\ > 0.046\ cfs$$

The maximum release rate is 0.069 cfs.

Step 5: Use the controlled-flow orifice equation to determine an appropriate orifice area by assuming the active storage depth.

In order to minimize the area required for the detention tank, choose the maximum depth that is still feasible according to site limitations and use a re-entrant orifice. In this case, the designer has chosen an active storage depth of 4 ft.

$$Q_O = C_D * A_o * \sqrt{2gH}$$

where:

Q_O = maximum release rate of orifice (cfs) = 0.069 cfs

C_D = coefficient of discharge, 0.52 for re-entrant orifice

A_O = area of orifice (sf)

g = acceleration due to gravity, 32.2 (ft/s^2)
H = maximum hydraulic head above the centerline of the orifice (ft) = 4 ft

$$0.069\ cfs = 0.52 * A_o * \sqrt{2 * 32.2\ \left(\frac{ft}{s^2}\right) * 4\ ft}$$

$$A_o = 0.008\ sf$$

Step 6: Translate the area of the controlled-flow orifice (A_O) into a diameter and check that it is greater than the minimum diameter of 1 in.

$$A_o = \frac{\left[\pi * \left(\frac{D_o}{2}\right)^2\right]}{144}$$

where:
A_O = area of orifice (sf) = 0.008 sf
D_O = diameter of orifice (in)

$$0.008\ sf = \frac{\left[\pi * \left(\frac{D_O}{2}\right)^2\right]}{144}$$

$$D_O = 1.21\ in\ > 1\ in \qquad OK$$

Set the orifice diameter to the nearest 0.25-inch interval rounding down, with a minimum orifice diameter of one-inch. In this case, use an orifice diameter of 1 inch.

Step 7: Confirm the orifice area of the selected orifice diameter from Step 6.

$$A_o = \frac{\left[\pi * \left(\frac{D_o}{2}\right)^2\right]}{144}$$

where:
A_O = area of orifice (sf)
D_O = diameter of orifice (in) = 1 inch

$$A_O = \frac{\left[\pi * \left(\frac{1\ in}{2}\right)^2\right]}{144}$$

$$A_O = 0.005\ sf$$

Step 8: Confirm the required active storage depth in the tank using the orifice area from Step 7.

$$Q_O = C_D * A_o * \sqrt{2gH}$$

where:

Q_O = maximum release rate of orifice (cfs) = 0.069 cfs

C_D = coefficient of discharge, 0.52 for re-entrant orifice

A_O = area of orifice (sf) = 0.005 sf

g = acceleration due to gravity, 32.2 (ft/s^2)

H = maximum hydraulic head above the centerline of the orifice (ft)

$$0.069\ cfs = 0.52 * 0.005\ sf * \sqrt{2 * 32.2\ \left(\frac{ft}{s^2}\right) * H}$$

$$H = 10.9\ ft$$

If the active storage depth is too high, then increase the orifice size by 0.25 inches and re-run Steps 7-8 until a suitable depth is identified. If the active storage depth is too low, then decrease the orifice size by 0.25 inches (but not less than 1 inch) and re-run Steps 7-8. Alternatively, the designer can choose a different orifice configuration as needed to modify the active storage depth.

In this case, the depth is too high to drain via gravity connection to the storm sewer. Using an orifice size of 1.25 inches results in an active storage depth of 3.4 ft.

Step 9: Set the dimensions of the detention tank's active storage zone.

Based on the active storage depth of 3.4 ft and the V_V of 229 cf, set the interior detention tank dimensions to L: 8.5 ft and W: 8.5 ft. The resulting detention tank has an active storage volume of 246 cf. Note that the exterior dimensions of the detention tank will be larger than the dimensions of the active storage zone (8.5'L x 8.5'W x 3.4'D) to accommodate wall thickness, bypass structures, and/or other internal features.

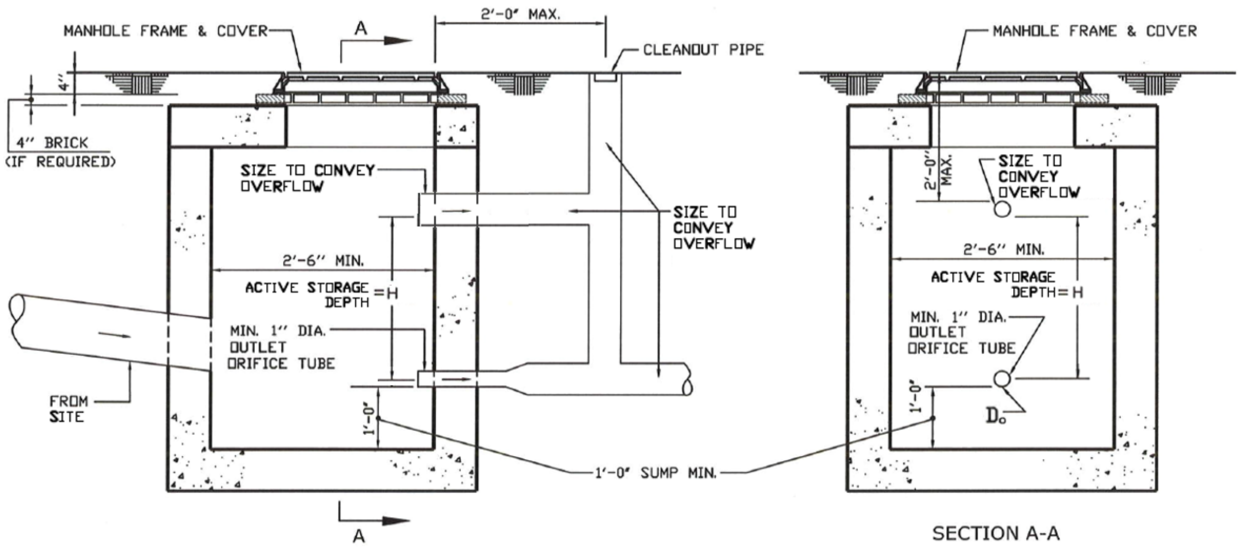


Figure F.5. Detention Tank with Re-Entrant Orifice

APPENDIX E

Site Design Example

Site Design Example

Design stormwater management practices for a 21,545 square foot commercial development that proposes a new site connection. This site is located within the sewershed of a combined sewer system and has no site constraints. Based on geotechnical investigations, the soil permeability rate across the site is at least 0.5 in/hr.

Step 1: Determine applicable permit requirements for the site.

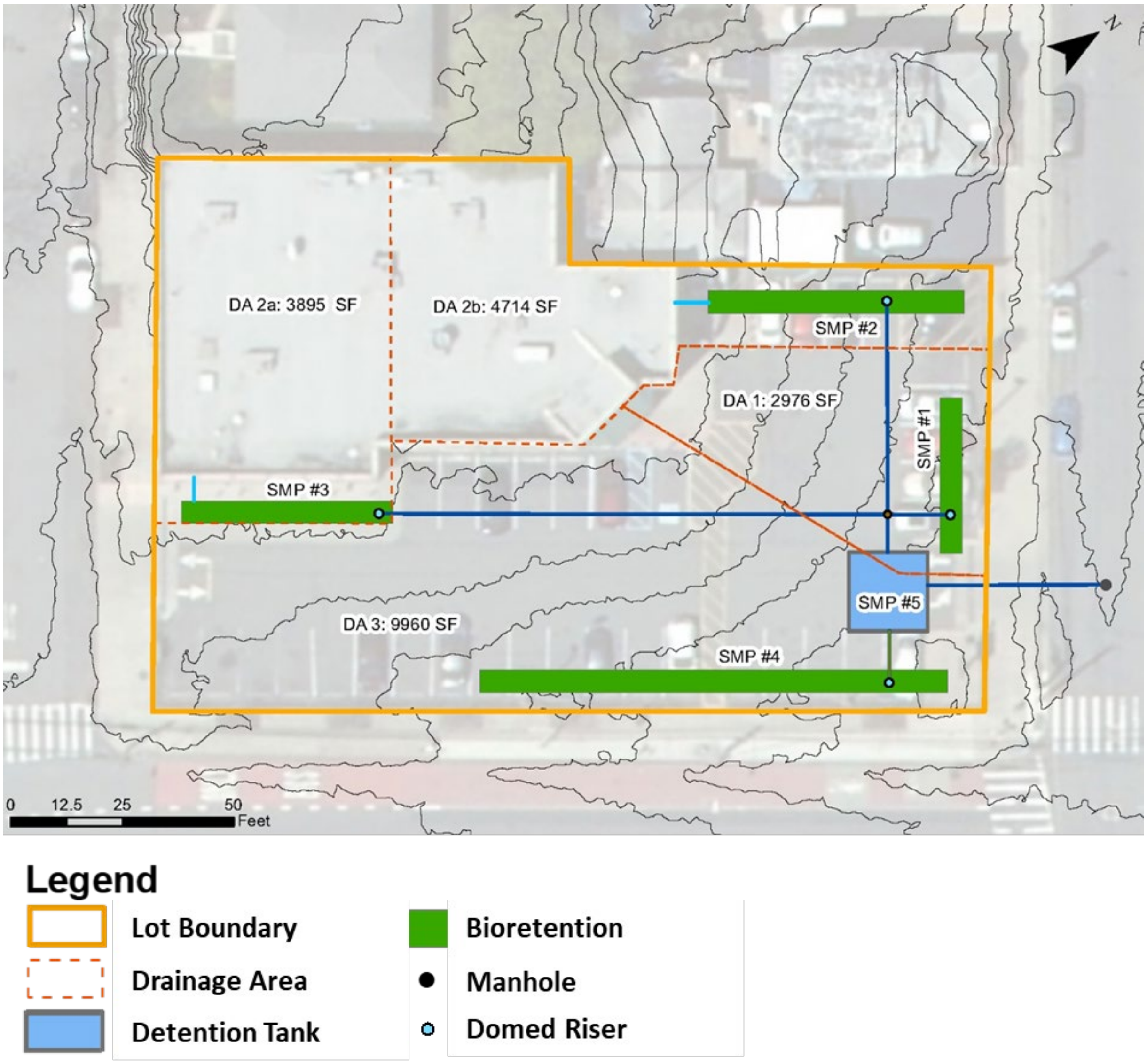
Since the project disturbs more than 20,000 square feet and involves commercial development, a Stormwater Construction Permit is applicable. As shown in Table 2.3 of Chapter 2, commercial development is a covered development activity that requires the preparation of a SWPPP meeting erosion and sediment control (ESC), water quality (WQ_V), and runoff reduction (RR) requirements. The no-net increase (NNI) requirement is not applicable because the project is not located in an MS4 sewershed area and does not discharge into an impaired water body.

The project proposes a new site connection and is located within the sewershed of a combined sewer system. Therefore, a Site Connection Permit is also applicable. A connection proposal must be prepared to meet the sewer operations (V_V) requirements.

Step 2: Use Appendix C to select appropriate practices for meeting the WQ_V, RR, and V_V requirements. The ESC requirements should be met using best practices in accordance with the NYS Standards and Specifications for Erosion and Sediment Control (The Blue Book).

Since the site has no constraints and the soil permeability rate is at least 0.5 in/hr, an infiltration practice is preferred. To meet the WQ_V and RR requirements, the designer has chosen to use a bioretention practice for each of the four drainage areas. The designer has chosen to use a detention tank to meet the V_V requirements.

Figure G.1. Schematic of Scenario 1



SMP 1: Bioretention

Design a bioretention practice (SMP 1) that will treat the water quality volume from an impervious area of 2,976 square feet with a runoff coefficient of 0.95. This example assumes a soil media saturated hydraulic conductivity of 2 in/hr, and an infiltration rate of 1.5 in/hr.

Note: If a bioretention practice is designed to meet the water quality volume, the practice will, by default, also meet the runoff reduction criteria.

Step 3.1: Calculate the WQ_V.

$$WQ_V = \frac{1.5\ in}{12} * A * R_V$$

where:
WQ_V = water quality volume (cf)
A = contributing area (sf) = 2,976 sf
R_V = runoff coefficient relating total rainfall and runoff
R_V = 0.05 + 0.009(I) = 0.95
I = percent impervious cover = 100%

$$WQ_V = \frac{1.5\ in}{12} * 2{,}976\ sf * 0.95$$

$$WQ_V = 353.4\ cf$$

Step 3.2: Calculate the minimum SMP area using the maximum loading ratio of 1:20 for a bioretention practice. Use the minimum area to set the initial length and width of the practice.

$$A_{SMP} = \frac{A}{20}$$

where:
A_{SMP} = area at the base of infiltration SMP (sf)
A = contributing area (sf) = 2,976 sf

$$A_{SMP} = \frac{2{,}976\ sf}{20}$$

$$A_{SMP} = 148.8\ sf$$

Round the SMP area up to 150 sf. Assume a 30 ft by 5 ft practice.

Step 3.3: Calculate the volume of surface ponding assuming the maximum surface ponding depth of 1 ft for a bioretention practice.

Assume the ponding zone is relatively flat.

$$V_P = A_{SMP} * D_P$$

where:
V_P = volume of surface ponding (cf)
A_{SMP} = area of the SMP (sf) = 150 sf
D_P = depth of ponding (ft) = 1 ft

$$V_P = 150\ sf * 1\ ft$$

$$V_P = 150\ cf$$

Since the bioretention practice uses engineered soil media, confirm that the volume of surface ponding is at least 10% of the water quality volume.

$$V_P = 150\ cf > 10\%\ of\ WQ_V = 35.3\ cf \qquad OK$$

In this case, the designer has also chosen to use a hydraulic connection between the ponding zone and the stone base. Therefore, the ponding zone does not need to temporarily store 75% of the water quality volume.

Step 3.4: Calculate the volume of voids in the soil media layer assuming a soil media depth of 2.5 ft, equal to the minimum soil media depth of 2.5 ft for a bioretention practice.

$$V_S = A_{SMP} * D_S * n_S$$

V_S = volume of voids in the soil media layer (cf)
A_{SMP} = area of the SMP (sf) = 150 sf
D_S = depth of soil media layer (ft) = 2.5 ft
n_S = available porosity of soil media (cf/cf) = 0.2 cf/cf

$$V_S = 150\ sf * 2.5\ ft\ * 0.2 \frac{cf}{cf}$$

$$V_S = 75\ cf$$

Step 3.5: Calculate the volume of voids created by internal structures.

Assume there are no internal structures in this bioretention practice, so the volume is 0.

$V_I = 0\ cf$

Step 3.6: Calculate the volume of voids in the drainage layer assuming a drainage media depth of 2.5 ft, which is greater than the minimum drainage media depth of 1 ft for a bioretention practice.

$$V_D = (A_{SMP} * D_D - V_{I,d}) * n_D$$

where:
V_D = volume of voids in the drainage layer (cf)
A_{SMP} = area of the SMP (sf) = 150 cf
D_D = depth of the drainage layer (ft) = 2.5 ft
$V_{I,d}$ = volume of voids created by internal structures within the drainage layer (cf) = 0 cf
n_D = porosity of drainage layer media (cf/cf) = 0.4 cf/cf

$$V_D = (150\ sf * 2.5\ ft - 0\ cf) * 0.4 \frac{cf}{cf}$$

$$V_D = 150\ cf$$

Step 3.7: Calculate the total SMP volume from the individual component volumes and compare to the WQ_V.

$$V_{SMP} = V_P + V_S + V_I + V_D$$

where:
V_{SMP} = storage volume of SMP (cf)
V_P = volume of surface ponding (cf) = 150 cf
V_S = volume of voids in the soil media layer (cf) = 75 cf
V_I = volume of voids created by internal structures such as chambers or pipes (cf) = 0 cf
V_D = volume of voids in the drainage layer (cf) = 150 cf

$$V_{SMP} = 150\ cf + 75\ cf + 0\ cf + 150\ cf$$

$$V_{SMP} = 375\ cf > WQ_V = 353.4\ cf \qquad OK$$

Step 3.8: Check that the ponding and infiltration drawdown times of the practice do not exceed the required times of 24 hours and 48 hours, respectively.

Infiltration drawdown time:

$$dt_{SMP} = \frac{V_{SMP}}{\left(\frac{i}{12}\right) * A_{SMP}}$$

where:

dt_{SMP} = drawdown time of infiltration SMP (hr)

V_{SMP} = volume of infiltration SMP (cf) = WQ_V = 375 cf

i = field measured infiltration rate (in/hr) = 1.5 in/hr

A_{SMP} = area at the base of infiltration SMP (sf) = 150 sf

$$dt_{SMP} = \frac{375\ cf}{\left(\frac{1.5\ in/hr}{12}\right) * 150\ sf}$$

$$dt_{SMP} = 20\ hr < 48\ hr \qquad OK$$

Surface ponding drawdown time:

$$dt_P = \frac{V_P}{\left(\frac{K_S}{12}\right) * \left(1 + \frac{0.5D_p}{D_m}\right) * \left(\frac{A_{P1} + A_{P2}}{2}\right)}$$

where:

dt_P = drawdown time of surface ponding (hr)

V_P = volume of surface ponding (cf) = 150 cf

K_S = saturated hydraulic conductivity of media below the surface ponding area (in/hr) = 2 in/hr

D_p = maximum depth of ponding (ft) = 1 ft

D_m = depth of media below surface ponding area (ft) = 2.5 ft

A_{P1} = area at the base of surface ponding zone (sf) = 150 sf

A_{P2} = area at the top of surface ponding zone (sf) = 150 sf

$$dt_P = \frac{150\ cf}{\left(\frac{2\frac{in}{hr}}{12}\right) * \left(1 + \frac{0.5 * 1\ ft}{2.5\ ft}\right) * \left(\frac{150\ sf + 150\ sf}{2}\right)}$$

$$dt_P = 5\ hr < 24\ hr \qquad OK$$

SMP 2-4: Bioretention

Steps 4-6: Design bioretention practices (SMP 2, SMP 3, and SMP 4) for the other three drainage areas by running through the same steps as for SMP 1. Assume a soil media saturated hydraulic conductivity of 2 in/hr, and an infiltration rate of 1.5 in/hr.

Table G.1 shows the final dimensions, SMP volume, and required water quality volume for each bioretention practice.

Table G.1. Summary of WQ_V Design

SMP #	Drainage Area (sf)	Dimensions (L' x W' x D')	SMP Volume (cf)	WQ_V (cf)
1	2,976	30 x 5 x 6	375	353.4
2	4,714	48 x 5 x 6	600	559.8
3	3,895	39 x 5 x 6	487.5	462.5
4	9,960	100 x 5 x 6	1,250	1,182.8

SMP 5: Detention Tank

Design a detention tank (SMP 5) that will treat the sewer operations volume from an impervious area of 21,545 square feet with a weighted runoff coefficient of 0.88.

Step 7.1: Identify the rainfall depth (R_D) based on the sewershed type and connection proposal type for the project. Use Table 2.7 in Chapter 2.

As determined in Step 1, the project requires a site connection permit (SCP). In addition, the project is located within the sewershed of a combined sewer system.

Table 2.7. Applied rainfall depth by sewershed type and connection proposal type.

R_D	Description
1.85	CSS areas with SCP
1.50	CSS areas with HCP
1.50	MS4 areas wiith SCP
1.10	MS4 areas with HCP

According to Table 2.7, $R_D = 1.85\ in.$

Step 7.2: Calculate the total V_V.

$$V_V = \frac{R_D}{12} * A * C_W$$

where:
V_V = sewer operations volume (cf)
R_D = rainfall depth (in) = 1.85 in
A = contributing area (sf) = 21,545 sf
C_W = weighted runoff coefficient relating peak rate of rainfall and runoff = 0.88

$$V_V = \frac{1.85\ in}{12} * 21{,}545\ sf * 0.88$$

$$V_V = 2{,}922.9\ cf$$

Step 7.3: Subtract the amount of SMP volume that may be credited towards meeting the total V_V from Step 7.2. The remaining volume ($V_{V,Tank}$) must be managed by the detention tank.

50% of the V_{SMP} from each bioretention practice can be credited towards the V_V.

Total creditable V_{SMP}:

$$V_{SMP,TC} = 0.5(V_{SMP,1} + V_{SMP,2} + V_{SMP,3} + V_{SMP,4})$$

where:
$V_{SMP,TC}$ = total creditable SMP volume (cf)
$V_{SMP,1}$ = volume from SMP 1 (cf) = 375 cf
$V_{SMP,2}$ = volume from SMP 2 (cf) = 600 cf
$V_{SMP,3}$ = volume from SMP 3 (cf) = 487.5 cf
$V_{SMP,4}$ = volume from SMP 4 (cf) = 1,250 cf

$$V_{SMP,TC} = 0.5(375\ cf + 600\ cf + 487.5\ cf + 1{,}250\ cf)$$

$$V_{SMP,TC} = 1{,}356.25\ cf$$

Remaining volume managed by the detention tank:

$$V_{V,Tank} = 2{,}922.9\ cf - 1{,}356.25\ cf$$

$$V_{V,Tank} = 1{,}566.65\ cf$$

Step 7.4: Calculate the release rate to be maintained by the controlled-flow orifice. Use the maximum release rate per acre (q) shown in Table 2.9, Chapter 2.

The project is located within the sewershed of a combined sewer system.

Table 2.9. Maximum release rate per acre (cfs/acre) by sewershed type.

q (cfs/acre)	**Description**
1.0	MS4 areas
0.1	CSS areas

According to Table 2.9, $q = 0.1\frac{cfs}{acre}$.

$$Q_{DRR} = \frac{q\ * A}{43560}\ or\ 0.046\ \ [whichever\ is\ greater]$$

where:
Q_{DRR} = maximum release rate for the site (cfs)
q = maximum release rate per acre (cfs/acre) = 0.1 cfs/acre
A = contributing area (sf) = 93,200 sf

$$Q_{DRR} = \frac{0.1\ \frac{cfs}{acre} * 21{,}545\ sf}{43560} or\ 0.046\ \ [whichever\ is\ greater]$$

$$Q_{DRR} = 0.049\ cfs\ > 0.046\ cfs$$

The maximum release rate is 0.049 cfs.

Step 7.5: Use the controlled-flow orifice equation to determine an appropriate orifice area by assuming the active storage depth.

In order to minimize the area required for the detention tank, choose the maximum depth that is still feasible according to site limitations and use a re-entrant orifice. In this case, the designer has chosen an active storage depth of 4 ft.

$$Q_O = C_D * A_o * \sqrt{2gH}$$

where:
Q_O = maximum release rate of orifice (cfs) = 0.049 cfs
C_D = coefficient of discharge, 0.52 for re-entrant orifice
A_O = area of orifice (sf)
g = acceleration due to gravity, 32.2 (ft/s^2)
H = maximum hydraulic head above the centerline of the orifice (ft) = 4 ft

$$0.049\ cfs = 0.52 * A_o * \sqrt{2 * 32.2\ \left(\frac{ft}{s^2}\right) * 4\ ft}$$

$$A_o = 0.006\ sf$$

Step 7.6: Translate the area of the controlled-flow orifice (A_O) into a diameter and check that it is greater than the minimum diameter of 1 in.

$$A_o = \frac{\left[\pi * \left(\frac{D_o}{2}\right)^2\right]}{144}$$

where:
A_O = area of orifice (sf) = 0.006 sf
D_O = diameter of orifice (in)

$$0.006\ sf = \frac{\left[\pi * \left(\frac{D_O}{2}\right)^2\right]}{144}$$

$$D_O = 1.05\ in\ > 1\ in \qquad OK$$

Set the orifice diameter to the nearest 0.25-inch interval rounding down, with a minimum orifice diameter of one-inch. In this case, use an orifice diameter of 1.00 inch.

Step 7.7: Confirm the orifice area of the selected orifice diameter from Step 7.6.

$$A_o = \frac{\left[\pi * \left(\frac{D_o}{2}\right)^2\right]}{144}$$

where:

A_O = area of orifice (sf)

D_O = diameter of orifice (in) = 1 in

$$A_O = \frac{\left[\pi * \left(\frac{1\ in}{2}\right)^2\right]}{144}$$

$$A_O = 0.005\ sf$$

Step 7.8: Confirm the required active storage depth in the tank using the orifice area from Step 7.7.

$$Q_O = C_D * A_o * \sqrt{2gH}$$

where:

Q_O = maximum release rate of orifice (cfs) = 0.049 cfs

C_D = coefficient of discharge, 0.52 for re-entrant orifice

A_O = area of orifice (sf) = 0.005 sf

g = acceleration due to gravity, 32.2 (ft/s^2)

H = maximum hydraulic head above the centerline of the orifice (ft)

$$0.049\ cfs = 0.52 * 0.005\ sf * \sqrt{2 * 32.2\ \left(\frac{ft}{s^2}\right) * H}$$

$$H = 5.5\ ft$$

If the active storage depth is too high, then increase the orifice size by 0.25 inches and re-run Steps 7.7-7.8 until a suitable depth is identified. If the active storage depth is too low, then decrease the orifice size by 0.25 inches (but not less than 1 inch) and re-run Steps 7.7-7.8. Alternatively, the designer can choose a different orifice configuration as needed to modify the active storage depth.

In this case, the depth is feasible.

Step 7.9: Set the dimensions of the detention tank's active storage zone.

Based on the active storage depth of 5.5 ft and the $V_{V,Tank}$ of 1,566.65 cf, set the interior detention tank dimensions to L: 17 ft and W: 17 ft. The resulting detention tank has an active storage volume of 1,589.5 cf. Note that the exterior dimensions of the detention tank will be larger than the dimensions of the active storage zone (17'L x 17'W x 5.5'D) to accommodate wall thickness, bypass structures, and/or other internal features.

Table G.2 summarizes the final designs for the bioretention practices and the detention tank.

Table G.2. Summary of WQ_V and V_V Design

SMP #	Drainage Area (sf)	Dimensions (L' x W' x D')	SMP Volume (cf)	WQ_V (cf)	V_V (cf)
1	2,976	30 x 5 x 6	375	353.4	187.5
2	4,714	48 x 5 x 6	600	559.8	300
3	3,895	39 x 5 x 6	487.5	462.5	243.75
4	9,960	100 x 5 x 6	1,250	1,182.8	625
5	21,545	17 x 17 x 5.5	1,589.5	0	1,589.5
Total	**21,545**	**-**	**-**	**2,558.5**	**2,945.75**

APPENDIX F

Controlled-Flow Pump Workbook

APPENDIX F: Detention in Series Workbook and Example

Workbook notes

This workbook is intended for detention system designs where a controlled-flow orifice with gravity drainage is infeasible. In these cases, the required maximum release rates are maintained with a controlled-flow pump system. When working to target a specific release rate for a pump system, the average release rate should not exceed the maximum rate for the facility at the time when the volume is being provided. The average rate for the system is determined by taking the average of the maximum and minimum rates for the system. The maximum and minimum rates are determined by finding the operation point for each rate respectively. The operation point is defined as the point where the system head curve intersects with the pump curve. The following steps and inputs are required to use this workbook.

The ***Notes*** section provides space to include notes and details on the system specifications. The ***Reviewer*** name should also be included here. Notes on how to choose a Hazen-Williams coefficient are also included in this section for reference.

Pump Head Losses

Reviewer:

Date: 4/2/2021

Notes:

****Choosing Hazen-Williams Coefficient:***
New Wrought or Cast Iron, Steel, Ductile Iron, Vitrified: 130
New Concrete: 120
Old Concrete or Brick: 100

The ***first section (1)*** asks for user inputs on the fittings proposed for the system and the number of each type of fitting. The user should place numbers corresponding to each type of fitting in the orange 'input' boxes. Any fitting that is not used can remain blank.

1. Input number of fittings in system.

Fittings	*Losses*	*# in System*
Strainer	320	
Globe Valve, Open	340	
Angle Valve, Open	170	
Swing Check Valve, Open	80	
Gate Valve, Open	7	
Ball Valve, Open	4	
Standard Elbow	32	
Medium Sweep Elbow	27	
Long Sweep Elbow	20	
45° Elbow	15	
Flow through Wye	30	
Tee - Flow thru Run	20	
Standard Tee - Side to Run	65	
Tee - Side to Run, With Throat	45	
Enlargement, d/D = 1/4	32	
Enlargement, d/D = 1/2	20	
Enlargement, d/D = 3/4	7	
Contraction, d/D = 1/4	15	
Contraction, d/D = 1/2	12	
Contraction, d/D = 3/4	7	

2. Input design information.

Pump start level		ft
Pump stop level		ft
Force main discharge elevation		ft
Detention volume		ft^3
Detention tank footprint		ft^2
Force main diameter		in
Force main length		ft
Hazen-Williams coefficient		--

The following information should be included in the ***second section (2).***

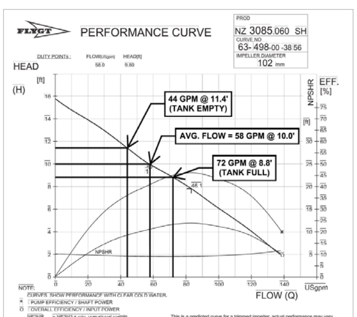
Pump start level (L3) - The elevation of water where the pump system is designed to turn on. This is typically near the top of the tank.
Pump stop level (L4) - The elevation of water where the pump system is designed to turn off. This is typically near the bottom of the tank.
Force main discharge elevation (L5) - The elevation that the proposed force main will discharge by gravity only (where it is no longer under pressure). The nature of this location requires that it be above the sewer.
Detention volume (L6) - The required detention volume in cubic feet calculated for the system. The required detention volume for singular detention systems can be computed using equations in Chapter 2, while the required detention volume of systems in series can be computed using equations in Section 4.11.
Detention tank footprint (L7) - The area of the detention tank in square feet.
Force main diameter (L8) - The force main pipe diameter in inches. Minimum of 2-inches and provided in half-inch increments.
Force main length (L9) - The force main length in feet, not including any equivalent lengths provided in section 1 (number of fittings in system).
Hazen-Williams coefficient (L10) - The proposed Hazen-Williams coefficient, typically 130 for new wrought or cast iron, steel, ductile iron, or vitrified clay pipes.

The ***third section (3)*** asks the user to recreate a pump curve provided by a manufacturer. The following table should be populated with values corresponding to a provided pump curve (example shown). The pump curve example given corresponds to ***Example 1.*** A table is also available for the user to input pump product information directly in the workbook.

Manfacturer:	
Duty Point: Flow (gpm):	
Duty Point: Head (ft):	
Product:	
Curve Number:	
Impeller Diameter (mm):	

3. Build pump curve (from manufacturer).

Head (ft)	*Flow (gpm)*



In the ***fourth section (4),*** the user is asked to provide the proposed maximum pump rate at which the pump will operate in gallons per minute ***(Q21)***, or the operation point for when the pressure head is the lowest, and the proposed minimum pump rate at which the pump will operate ***(Q22)***, or the operation point for when the pressure head is the highest. These inputs will allow for the following outputs to be calculated.

The equivalent length of pipe for fittings (V15)
The minimum static lift in feet (V17)
The maximum static lift in feet (V18)
The provided storage depth in feet (V19)
The maximum water level in feet (V20)
The maximum head loss in feet (P21)
The minimum head loss in feet (P22)
The maximum pump rate in cubic feet-per-second (V21)
The minimum pump rate in cubic feet-per-second (V22)
The average pump rate in cubic feet-per-second (V23), which is the average of V21 and V22.

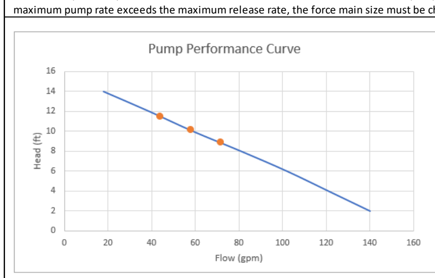
4. Change minimum and maximum flow rates until points align with pump curve.

	Head (ft)	*Flow (gpm)*
Maximum	8.81	72
Minimum	11.44	44
Average	10.12	58

Calculations:

Equivalent length of pipe for fittings	152	ft
Minimum static lift	5.01	ft
Maximum static lift	9.92	ft
Provided storage depth	5.24	ft
Maximum water level	156.99	ft
Maximum pump rate	0.160	cfs
Minimum pump rate	0.098	cfs
Average rate	0.129	cfs
$V^2/2g$	0.17	ft

The process of finding the actual pump behavior requires testing a proposed maximum and minimum pump rate (***Q21*** and ***Q22***) against the minimum and maximum head losses (***P21*** and ***P22***) - iterating until each operation point is found. The maximum and minimum rates must be adjusted until agreement with the pump curve is achieved. Once each operation point is found, this determines the actual release rate of the pump system. If this is lower than the maximum, then the pump system is acceptable. If the maximum pump rate exceeds the maximum release rate, the force main size must be changed or a smaller pump should be selected.



Space for additional calculations is provided in case they are required for specific design systems.

Flow Rate

1. Input values:	Pipe diameter		in
	Hazen-Williams C		--
	Pipe length		ft
	Head loss		ft
2. Results:	*Flow rate Q*	#DIV/0!	cfs
		#DIV/0!	gpm
	$V^2/2g$	#DIV/0!	ft

Head Loss

1. Input values: (**Flow rate can be entered in either cfs or gpm, or both.)	Pipe diameter		in
	Hazen-Williams C		--
	Flow rate Q**		cfs
			gpm
	Pipe length		ft
2. Results:	*Head loss % (Q, cfs)*	#DIV/0!	%
	Head loss % (Q, gpm)	#DIV/0!	%
	Head loss	#DIV/0!	ft
	$V^2/2g$	#DIV/0!	ft

Equivalent Pipe Length (Pipes in Parallel)

1. For two pipes in parallel to be replaced by a single pipe of equivalent capacity, input:	Pipe 1 diameter		in
	Pipe 1 length		ft
	Pipe 2 diameter		in
	Pipe 2 length		ft
	Assumed head loss		ft
	Diameter of proposed eq. pipe		in
2. Equivalent pipe length:		#DIV/0!	ft

Equivalent Pipe Length (Pipes in Series)

1. For two pipes connected in series to be replaced by a single pipe of equivalent capacity, input:	Pipe 1 diameter		in
	Pipe 1 length		ft
	Pipe 2 diameter		in
	Pipe 2 length		ft
	Assumed flow rate Q		cfs
	Diameter of proposed eq. pipe		in
2. Equivalent pipe length:		#DIV/0!	ft

Pump Head Losses

Reviewer:
Date: 11/24/2021

Notes:

Choosing Hazen-Williams Coefficient:
New Wrought or Cast Iron, Steel, Ductile Iron, Vitrified: 130
New Concrete: 120

*Key

#	*user input*
#	*provided pump curve information*
#	*calculation*

1. Input number of fittings in system.

Fittings	*Losses*	*# in System*
Strainer	320	
Globe Valve, Open	340	
Angle Valve, Open	170	
Swing Check Valve, Open	80	
Gate Valve, Open	7	
Ball Valve, Open	4	
Standard Elbow	32	
Medium Sweep Elbow	27	
Long Sweep Elbow	20	
45° Elbow	15	
Flow through Wye	30	
Tee - Flow thru Run	20	
Standard Tee - Side to Run	65	
Tee - Side to Run, With Throat	45	
Enlargement, d/D = 1/4	32	
Enlargement, d/D = 1/2	20	
Enlargement, d/D = 3/4	7	
Contraction, d/D = 1/4	15	
Contraction, d/D = 1/2	12	
Contraction, d/D = 3/4	7	

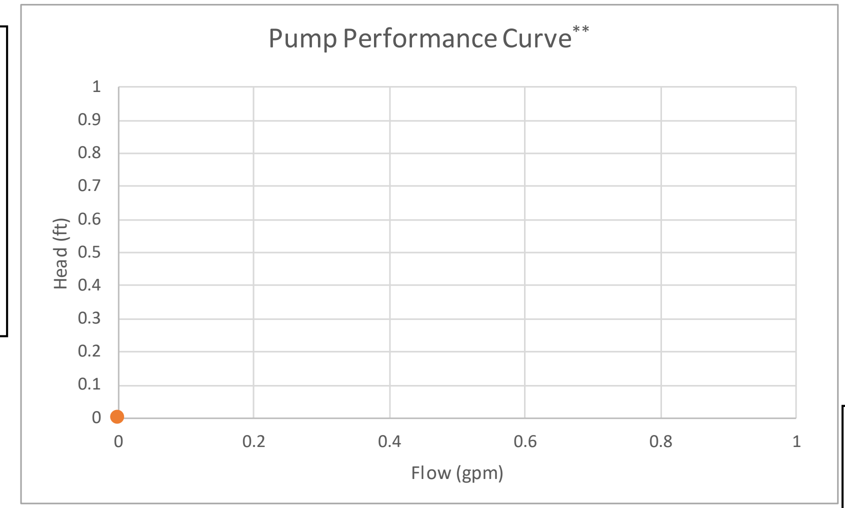
2. Input design information.

Pump start level		ft
Pump stop level		ft
Force main discharge elevation		ft
Detention volume		ft^3
Detention tank footprint		ft^2
Force main diameter		in
Force main length		ft
Hazen-Williams coefficient		--

3. Build pump curve (from manufacturer).

Head (ft)	*Flow (gpm)*

*Note that applicants must include the pump curve on the site plan, along with all of the necessary



Manfacturer:	
Duty Point: Flow (gpm):	
Duty Point: Head (ft):	
Product:	
Curve Number:	
Impeller Diameter (mm):	

4. Change minimum and maximum flow rates until points align with pump curve.

	Head (ft)	*Flow (gpm)*
Maximum	#DIV/0!	
Minimum	#DIV/0!	
Average	#DIV/0!	#DIV/0!

Calculations:

Equivalent length of pipe for fittings	0	ft
Minimum static lift	#DIV/0!	ft
Maximum static lift	0.00	ft
Provided storage depth	#DIV/0!	ft
Maximum water level	#DIV/0!	ft
Maximum pump rate	0.000	cfs
Minimum pump rate	0.000	cfs
Average rate	0.000	cfs
$V^2/2g$	#DIV/0!	ft

Flow Rate

1. Input values:	Pipe diameter		in
	Hazen-Williams C		--
	Pipe length		ft
	Head loss		ft
2. Results:	*Flow rate Q*	#DIV/0!	cfs
		#DIV/0!	gpm
	$V^2/2g$	#DIV/0!	ft

Head Loss

1. Input values: (**Flow rate can be entered in either cfs or gpm, or both.)	Pipe diameter		in
	Hazen-Williams C		--
	Flow rate Q**		cfs
			gpm
	Pipe length		ft
2. Results:	*Head loss % (Q, cfs)*	#DIV/0!	%
	Head loss % (Q, gpm)	#DIV/0!	%
	Head loss	#DIV/0!	ft
	$V^2/2g$	#DIV/0!	ft

Equivalent Pipe Length (Pipes in Parallel)

1. For two pipes in parallel to be replaced by a single pipe of equivalent capacity, input:	Pipe 1 diameter		in
	Pipe 1 length		ft
	Pipe 2 diameter		in
	Pipe 2 length		ft
	Assumed head loss		ft
	Diameter of proposed eq. pipe		in
2. Equivalent pipe length:		#DIV/0!	ft

Equivalent Pipe Length (Pipes in Series)

1. For two pipes connected in series to be replaced by a single pipe of equivalent capacity, input:	Pipe 1 diameter		in
	Pipe 1 length		ft
	Pipe 2 diameter		in
	Pipe 2 length		ft
	Assumed flow rate Q		cfs
	Diameter of proposed eq. pipe		in
2. Equivalent pipe length:		#DIV/0!	ft

Pump Head Losses

Reviewer: C. Moskos
Date: 11/24/2021

Example:
- 1 pump
- Fittings: 1 strainer, 1 swing check valve, 3 ball valves, 3 standards elbows, 1 45° elbow, 1 'flow thru run' tee, 1 'side to run' tee
- Pump start level: 152.08'
- Pump stop level: 151.75'
- Force main discharge elevation: 162'
- Detention volume: 1919 ft^3
- Detention tank footprint: 366 ft^2
- Force main diameter: 3"
- Force main length: 62'
- Hazen-Williams coefficient: 130*

Choosing Hazen-Williams Coefficient:
New Wrought or Cast Iron, Steel, Ductile Iron, Vitrified: 130
New Concrete: 120

1. Input number of fittings in system.

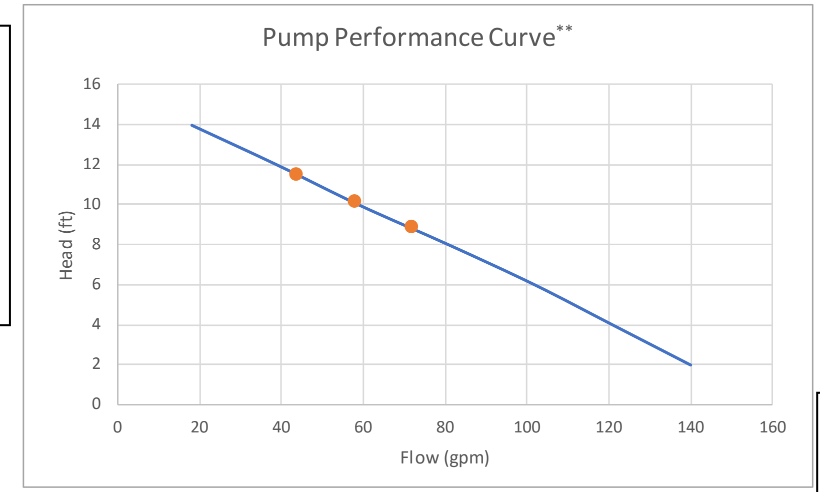
Fittings	*Losses*	*# in System*
Strainer	320	1
Globe Valve, Open	340	
Angle Valve, Open	170	
Swing Check Valve, Open	80	1
Gate Valve, Open	7	
Ball Valve, Open	4	3
Standard Elbow	32	3
Medium Sweep Elbow	27	
Long Sweep Elbow	20	
45° Elbow	15	1
Flow through Wye	30	
Tee - Flow thru Run	20	1
Standard Tee - Side to Run	65	1
Tee - Side to Run, With Throat	45	
Enlargement, d/D = 1/4	32	
Enlargement, d/D = 1/2	20	
Enlargement, d/D = 3/4	7	
Contraction, d/D = 1/4	15	
Contraction, d/D = 1/2	12	
Contraction, d/D = 3/4	7	

2. Input design information.

Pump start level	152.08	ft
Pump stop level	151.75	ft
Force main discharge elevation	162.00	ft
Detention volume	1919	ft^3
Detention tank footprint	366	ft^2
Force main diameter	3.0	in
Force main length	62.00	ft
Hazen-Williams coefficient	130	--

3. Build pump curve (from manufacturer).

Head (ft)	*Flow (gpm)*
2	140
4	121
6	102
8	81
10	59
12	39
14	18



Manfacturer:	Flygt
Duty Point: Flow (gpm):	58.0
Duty Point: Head (ft):	9.8
Product:	NZ 3085.060 SH
Curve Number:	63-498-00-3856
Impeller Diameter (mm)	102

4. Change minimum and maximum flow rates until points align with pump curve.

	Head (ft)	*Flow (gpm)*
Maximum	8.81	72
Minimum	11.44	44
Average	10.12	58

Calculations:

Equivalent length of pipe for fittings	152	ft
Minimum static lift	5.01	ft
Maximum static lift	9.92	ft
Provided storage depth	5.24	ft
Maximum water level	156.99	ft
Maximum pump rate	0.160	cfs
Minimum pump rate	0.098	cfs
Average rate	0.129	cfs
$V^2/2g$	0.17	ft

*Key

#	*user input*
#	*provided pump curve information*
#	*calculation*

****Note that applicants must include the pump curve on the site plan, along with all of the necessary***

Flow Rate

1. Input values:	Pipe diameter		in
	Hazen-Williams C		--
	Pipe length		ft
	Head loss		ft
2. Results:	*Flow rate Q*	#DIV/0!	cfs
		#DIV/0!	gpm
	$V^2/2g$	#DIV/0!	ft

Head Loss

1. Input values: (**Flow rate can be entered in either cfs or gpm, or both.)	Pipe diameter		in
	Hazen-Williams C		--
	Flow rate Q**		cfs
			gpm
	Pipe length		ft
2. Results:	*Head loss % (Q, cfs)*	#DIV/0!	%
	Head loss % (Q, gpm)	#DIV/0!	%
	Head loss	#DIV/0!	ft
	$V^2/2g$	#DIV/0!	ft

Equivalent Pipe Length (Pipes in Parallel)

1. For two pipes in parallel to be replaced by a single pipe of equivalent capacity, input:	Pipe 1 diameter		in
	Pipe 1 length		ft
	Pipe 2 diameter		in
	Pipe 2 length		ft
	Assumed head loss		ft
	Diameter of proposed eq. pipe		in
2. Equivalent pipe length:		#DIV/0!	ft

Equivalent Pipe Length (Pipes in Series)

1. For two pipes connected in series to be replaced by a single pipe of equivalent capacity, input:	Pipe 1 diameter		in
	Pipe 1 length		ft
	Pipe 2 diameter		in
	Pipe 2 length		ft
	Assumed flow rate Q		cfs
	Diameter of proposed eq. pipe		in
2. Equivalent pipe length:		#DIV/0!	ft

APPENDIX G

Detention in Series Workbook and Examples

APPENDIX G: Detention in Series Workbook and Examples

PROJECT NAME

PROJECT ID

ADDRESS

Notes: Use this form to determine the required storage volumes for detention systems in series. There are two parts to this form. In the first part, users input properties of the downstream detention system. In the second part, users input properties of each individual upstream area that drains to the downstream detention system. Inputs are entered in the yellow cells and outputs are shown in the gray cells.

ERRORS None

DOWNSTREAM SYSTEM

INPUTS			OUTPUTS	
Permit Type	Total Contributing Area	Maximum Release Rate	Required Detention Volume	Effective C-value
name	sf	cfs	cf	#
CSS - SCP	40000	0.092	3883	0.63

UPSTREAM SYSTEM

INPUTS					OUTPUTS	
TDA ID	TDA Area	C-value	Detention System Type	Maximum Release Rate	Required Detention Volume	Effective C-value
name	sf	#	name	cfs	cf	#
1	20000	0.95	Blue Roof	0.5	1829	0.41
2	20000	0.85	None			0.85

I. Detention Facilities with a variable outflow

For a detention facility where the outflow is controlled by means of an outlet orifice tube, subject to a head which increases as the depth of storage increases, in a storage facility with an approximately uniform area with respect to storage height, and for roof detention by means of controlled flow roof drains, the average flow rate out of the detention facility is approximately 2/3 of the maximum outflow rate. The following procedure is used to compute the maximum required detention storage volume in ft^3:

1. Compute the duration of the storm in minutes with a 10 yr. return frequency, t_V, which requires the maximum detention volume with outflow controlled by an orifice or by controlled flow roof drains, by the equation:

$$t_V = 0.27(C_{WT} A_t / Q_{DRR})^{0.5} - 15$$

where:
- t_V = the duration of the storm in min. with a 10 yr. return frequency requiring the maximum detention volume with a variable outflow
- C_{WT} = the weighted runoff coefficient for the area tributary to the detention facility
- A_t = the area tributary to the detention facility in ft^2
- Q_{DRR} = the detention facility maximum release rate in cfs

2. Compute the maximum required detention volume in ft^3 with outflow controlled by an orifice tube or by controlled flow roof drains, V_V, by the equation:

$$V_V = [0.19 C_{WT} A_t / (t_V + 15) - 40 Q_{DRR}]\, t_V$$

where:
- V_V = the maximum required detention volume in ft^3 with a variable outflow
- C_{WT} = the weighted runoff coefficient for the area tributary to the detention facility
- A_t = the area tributary to the detention facility in ft^2
- t_V = the duration of the storm in min., with a 10 yr. return frequency, requiring the maximum detention volume with a variable outflow
- Q_{DRR} = the detention facility maximum release rate in cfs

3. For roof detention compute the duration of the storm in minutes with a 10 yr. return frequency, t_V, which requires the maximum detention volume, and compute the maximum required detention volume in ft^3 with outflow controlled by controlled flow roof drains, V_V, with a detention facility maximum release rate in cfs, Q_{DRR}, as in I-1 through I-2 above. Confirm that the detention volume provided on the controlled flow roof, based on the slopes and geometry of the roof, is equal to or greater than the volume required, V_V, and that the actual release rate from the roof does not exceed the proposed maximum release rate in cfs for the roof, Q_{DRR}, following the procedure detailed in the DEP "Guidelines for the Design and Construction of Stormwater Management Systems".

III. Weighted effective runoff coefficient for series detention systems

1. A. When the flow from a roof which has been restricted by controlled flow roof drains is discharged to a subsurface detention facility, the weighted effective weighted runoff coefficient for the roof, C_{WE}, is based on the average rainfall intensity in in/hr, i_{10}, for the duration in min. of the storm with a 10 yr. return frequency, as computed under I-1, t_V, for which the roof detention volume in ft^3, V_V, was computed. Compute the weighted effective runoff coefficient for the roof with runoff restricted by controlled flow roof drains, C_{WE}, by the equation:

$$C_{WE} = 311 Q_{DRR}(t_V + 15) / A_t$$

where:
- C_{WE} = the weighted effective weighted runoff coefficient for the roof with runoff restricted by controlled flow roof drains
- Q_{DRR} = the maximum release rate from the roof in cfs
- t_V = the duration in min of the rainfall event for which the roof detention volume was computed
- A_t = the area of the roof in ft^2 tributary to the roof detention
- 311 = 43,560 ft^2 per ac./140

B. Use this effective weighted runoff coefficient for the roof with runoff restricted by controlled flow roof drains, C_{WE}, to compute the weighted runoff coefficient for the area tributary to the sub- surface detention facility, C_{WT}, to which this restricted roof will discharge, as on page 2 above. Compute the maximum required detention volume in ft^3, V_V as in I-1 through I-2, or V_C as in II-1 through II-2.

Detention System Types
None
Blue Roof
Tank
Subsurface
Pond
Wetland

Permit Type	Vv
CSS - SCP	1.85
MS4 - SCP	1.50
CSS - HCP	1.50
MS4 - HCP	1.10

Detention in Series Example

A site in Queens consists of a multistory office building and a parking lot for its tenants. The site was proposed to connect to a 15 in. combined sewer. The building owner intends to use a blue roof and detention tank in series to meet the stormwater management requirement. The total roof area will be used for detention. Design a blue roof and a downstream detention system that treats runoff from the roof and the parking lot, given the following:

Total Contributing Area = 40,000 sf
Roof (sloped 1/8 in per ft) = 20,000 sf @ 0.95 runoff coefficient.
Paved = 20,000 sf @ 0.85 runoff coefficient

Use the Detention In Series Workbook provided in Appendix I.

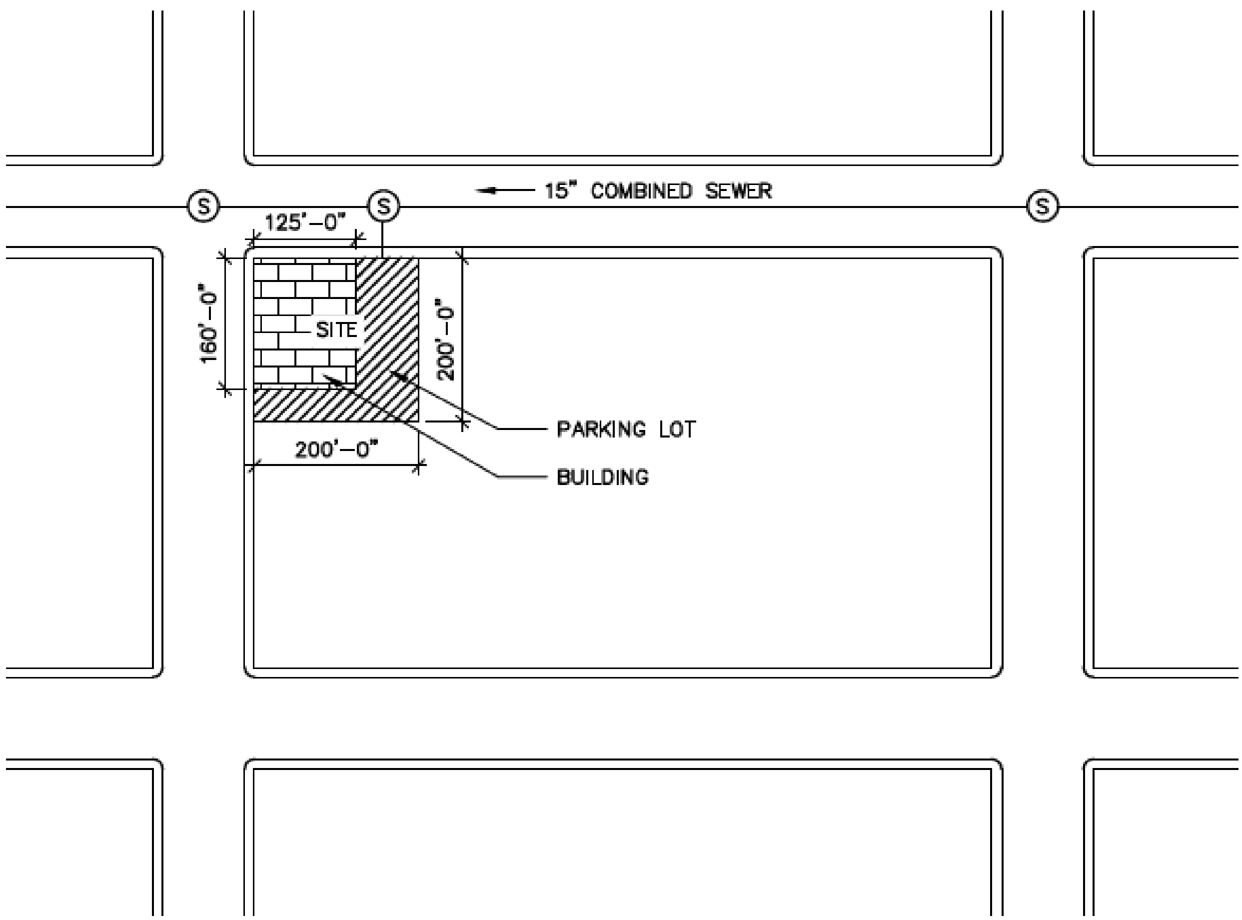


Figure I.1. Schematic of Example 1 (Not to Scale)

Step 1: Input the properties of the blue roof that will drain into the downstream detention system.

The first upstream area that drains to the downstream detention system is the 20,000 sf blue roof.

UPSTREAM SYSTEM						
INPUTS					OUTPUTS	
TDA ID	TDA Area	C-value	Detention System Type	Maximum Release Rate	Required Detention Volume	Effective C-value
name	sf	#	name	cfs	cf	#
1	20000	0.95	Blue Roof			

Figure I.2. Inputs for the Blue Roof Properties

Step 2: Design the maximum release rate to be maintained by the blue roof.

Identify a controlled-flow roof drain by an approved manufacturer. In this case, the designer has selected a controlled-flow roof drain that restricts flow to 10 gpm/in.

The roof has an area of 20,000 sf. According to the 2014 Plumbing Code by the NYC Department of Buildings, not less than four roof drains shall be installed in roofs over 10,000 sf in area. In this case, the designer has chosen to install four roof drains.

Ponding depths should not exceed 4 inches above the low point (or as specified in the current Construction Codes). The designer has chosen to use a ponding depth of 2 inches.

$$Q_{ROOF} = \frac{Q_i N_{RD} d_{max}}{449}$$

where:

Q_{ROOF} = maximum release rate from rooftop detention (cfs)

Q_i = maximum release rate from each drain (gpm/in) = 10 gpm/in

N_{RD} = number of roof drains = 4

d_R = the roof drain depth of flow (in) = 2 in

$$Q_{ROOF} = \frac{10\frac{gpm}{in} * 4 * 2\ in}{449}$$

$$Q_{ROOF} = 0.18\ cfs$$

The blue roof can maintain a maximum release rate of approximately 0.2 cfs. Input this maximum release rate into the workbook.

UPSTREAM SYSTEM						
INPUTS					OUTPUTS	
TDA ID	TDA Area	C-value	Detention System Type	Maximum Release Rate	Required Detention Volume	Effective C-value
name	sf	#	name	cfs	cf	#
1	20000	0.95	Blue Roof	0.2		

Figure I.3. Input for the Maximum Release Rate Maintained by the Blue Roof

Step 3: Based on the inputs from Steps 1 and 2, the workbook will automatically calculate the duration of a storm (min) with a 10-year return frequency. This calculation is shown below.

The total roof area will be used for detention. Therefore, the available area is the entire 20,000 sf.

$$t_V = 0.27(\frac{C_{WT}A_t}{Q_{DRR}})^{0.5} - 15$$

where:

t_V = the duration of the storm with a 10 yr. return frequency requiring the maximum detention volume with a variable outflow (min)

C_{WT} = the weighted runoff coefficient for the contributing area = 0.95

A_t = contributing area (sf) = 20,000 sf

Q_{DRR} = maximum release rate for the site (cfs) = 0.2 cfs

$$t_V = 0.27(\frac{0.95 * 20{,}000\ sf}{0.2\ cfs})^{0.5} - 15$$

$$t_V = 68.2\ min$$

Step 4: Based on the inputs from Steps 1 and 2, the workbook will automatically calculate the required detention volume through the blue roof. This calculation is shown below.

$$V_V = (\frac{0.19C_{WT}A_t}{t_V + 15} - 40Q_{DRR})t_V$$

where:

V_v = the maximum required detention volume (cf)

C_{WT} = the weighted runoff coefficient for the contributing area = 0.95

A_t = contributing area (sf) = 20,000 sf

t_V = the duration of the storm with a 10 yr. return frequency requiring the maximum detention volume with a variable outflow (min) = 68.2 min

Q_{DRR} = maximum release rate for the site (cfs) = 0.2 cfs

$$V_V = [\frac{0.19 * 0.95 * 20{,}000\ sf}{68.2\ min + 15} - (40 * 0.2\ cfs)](68.2\ min)$$

$$V_V = 2{,}414\ cf$$

UPSTREAM SYSTEM

INPUTS | OUTPUTS

TDA ID	TDA Area	C-value	Detention System Type	Maximum Release Rate	Required Detention Volume	Effective C-value
name	sf	#	name	cfs	cf	#
1	20000	0.95	Blue Roof	0.2	2414	

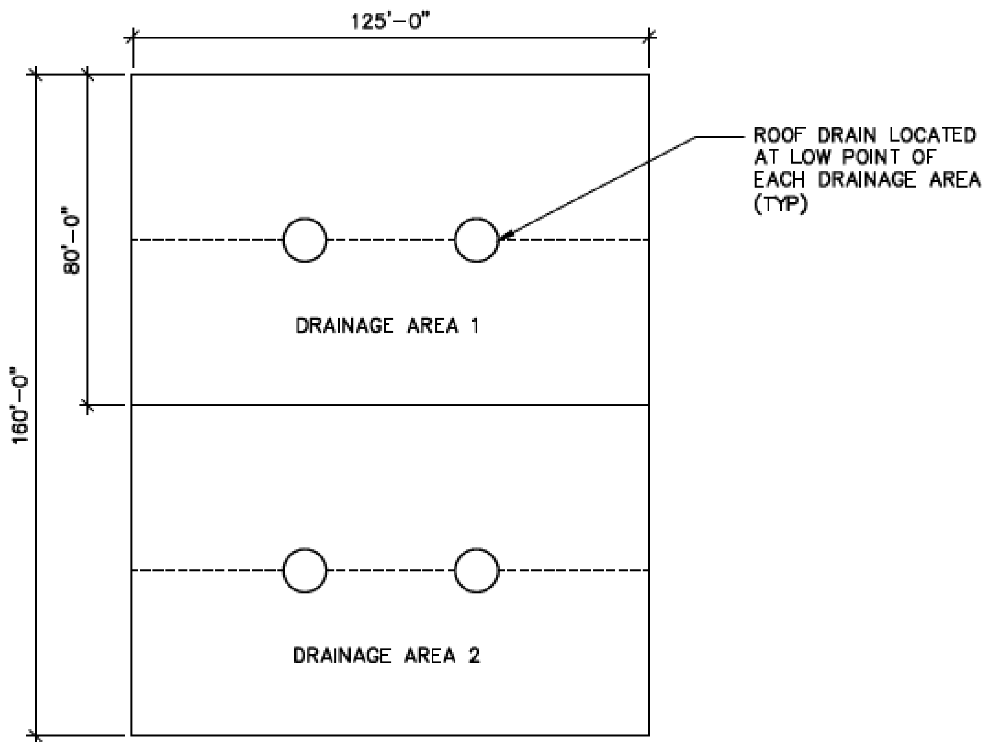
Figure I.4. Output for the Required Detention Volume Through the Blue Roof

Step 5: Check that the available storage volume of the roof is greater than the required detention volume.

The total roof area will be used for detention. Therefore, the available area is the entire 20,000 sf.

The designer has considered two different roof configurations: 1) a uni-directionally sloped roof, as shown in Figure I.5 and 2) a multi-directionally sloped roof, as shown in Figure I.6.

Uni-directionally Sloped Roof:



The lengths and widths of each drainage area are as follows:

Drainage Area 1: 125'L x 80'W
Drainage Area 2: 125'L x 80'W

If the roof is sloped 1/8 in per ft, the height difference between the high and low points of each drainage area is 5 inches. The ponding depth is 2 inches. Therefore, the high point of each drainage area will not be inundated.

Calculate the available storage volume of each drainage area, using the volume of a triangular prism.

$$V_A = \frac{1}{2} LW * \frac{d_R}{12}$$

where:
V_A = the available storage volume of each drainage area (cf)
L = the length of each drainage area (ft) = 125 ft
W = the width of each drainage area (ft = 80 ft
d_R = the roof drain depth of flow (in) = 2 in

$$V_A = \frac{1}{2} * 125\ ft * 80\ ft * \frac{2\ in}{12}$$

$$V_A = 833\ cf$$

The total available storage volume is:

$$V_T = V_1 + V_2$$

where:
V_T = the total available storage volume (cf)
V_1 = the available storage volume of Drainage Area 1 (cf) = 833 cf
V_2 = the available storage volume of Drainage Area 2 (cf) = 833 cf

$$V_T = 833\ cf + 833\ cf$$

$$V_T = 1{,}666\ cf \leq V_V = 2{,}414\ cf \qquad \boldsymbol{NOT\ MET}$$

Since the required detention volume is greater than the available storage volume, select a different controlled-flow roof drain or design depth of flow and re-run Steps 2-4.

In this case, the designer has chosen 3 inches as the new design depth of flow. The new ponding depth results in a maximum release rate of 0.27 cfs, a required detention volume of 2,242 cf, and a total available storage volume of 2,500 cf.

Multi-directionally Sloped Roof:

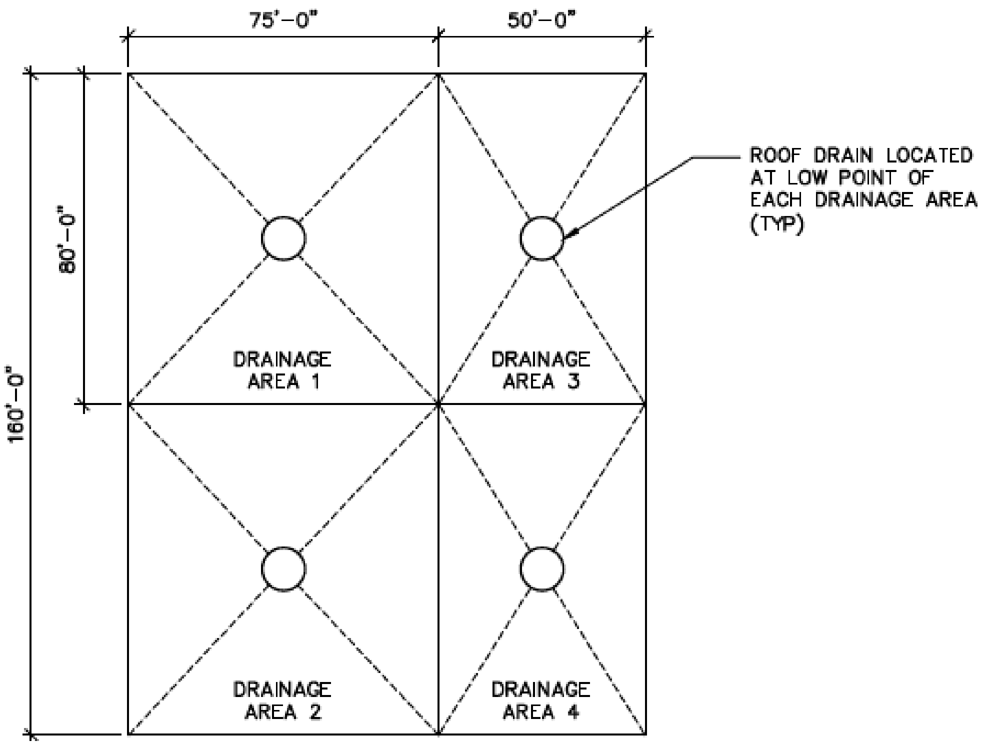


Figure I.6. Plan View of Multi-Directionally Sloped Blue Roof

The lengths and widths of each drainage area are as follows:

Drainage Area 1: 75'L x 80'W
Drainage Area 2: 75'L x 80'W
Drainage Area 3: 50'L x 80'W
Drainage Area 4: 50'L x 80'W

If the roof is sloped 1/8 in per ft, the height difference between the high and low points is 6.9 inches for drainage areas 1 and 2, and 5.9 inches for drainage areas 3 and 4. The ponding depth is 2 inches. Therefore, the high point of each drainage area will not be inundated.

Calculate the available storage volume of each drainage area, using the volume of a pyramid.

Drainage Areas 1 and 2:

$$V_A = \frac{1}{3} LW * \frac{d_R}{12}$$

where:
V_A = the available storage volume of each drainage area (cf)

W = the width of each drainage area (ft = 80 ft
d_R = the roof drain depth of flow (in) = 2 in

$$V_A = \frac{1}{3} * 75\ ft * 80\ ft * \frac{2\ in}{12}$$

$$V_A = 333\ cf$$

Drainage Areas 3 and 4:

$$V_A = \frac{1}{3} LW * \frac{d_R}{12}$$

where:
V_A = the available storage volume of each drainage area (cf)
L = the length of each drainage area (ft) = 50 ft
W = the width of each drainage area (ft = 80 ft
d_R = the roof drain depth of flow (in) = 2 in

$$V_A = \frac{1}{3} * 50\ ft * 80\ ft * \frac{2\ in}{12}$$

$$V_A = 222\ cf$$

The total available storage volume is:

$$V_T = V_1 + V_2 + V_3 + V_4$$

where:
V_T = the total available storage volume (cf)
V_1 = the available storage volume of Drainage Area 1 (cf) = 333 cf
V_2 = the available storage volume of Drainage Area 2 (cf) = 333 cf
V_3 = the available storage volume of Drainage Area 3 (cf) = 222 cf
V_4 = the available storage volume of Drainage Area 4 (cf) = 222 cf

$$V_T = 333\ cf + 333\ cf + 222\ cf + 222\ cf$$

$$V_T = 1{,}110\ cf \leq V_V = 2{,}414\ cf \qquad \boldsymbol{NOT\ MET}$$

Since the required detention volume is greater than the available storage volume, select a different controlled-flow roof drain or design depth of flow and re-run Steps 2-4.

In this case, the designer has chosen 3 inches as the new design depth of flow. The new ponding depth results in a maximum release rate of 0.27 cfs, a required detention volume of 2,242 cf, and a total available storage volume of 1,666 cf.

A uni-directionally sloped roof provides sufficient storage volume for a ponding depth of 3 inches. The multi-directionally sloped roof does not provide enough storage volume for the same depth. Therefore, the designer has chosen to use a uni-directionally sloped roof, with a ponding depth of 3 inches.

The inputs have been updated, and the workbook automatically outputs the new required detention volume of 2,242 cf.

UPSTREAM SYSTEM

INPUTS					OUTPUTS	
TDA ID	TDA Area	C-value	Detention System Type	Maximum Release Rate	Required Detention Volume	Effective C-value
name	sf	#	name	cfs	cf	#
1	20000	0.95	Blue Roof	0.27	2242	

Figure I.7. Inputs and Output for the Required Detention Volume Through the Blue Roof, Using a Ponding Depth of 3"

Step 6: Based on the inputs from Steps 1 and 2, the workbook will automatically calculate the effective weighted runoff coefficient for the blue roof. This calculation is shown below.

$$C_{WE} = \frac{311Q_{DRR}(t_V + 15)}{A_t}$$

where:

C_{WE} = the effective weighted runoff coefficient for the roof with runoff restricted by controlled-flow roof drains

Q_{DRR} = maximum release rate for the site (cfs) = 0.27 cfs

t_V = the duration of the storm with a 10 yr. return frequency requiring the maximum detention volume with a variable outflow (min) = 56.6 min

A_t = contributing area (sf) = 20,000 sf

$$C_{WE} = \frac{311 * 0.27\ cfs * (56.6\ min + 15)}{20{,}000\ sf}$$

$$C_{WE} = 0.301$$

UPSTREAM SYSTEM

INPUTS					OUTPUTS	
TDA ID	TDA Area	C-value	Detention System Type	Maximum Release Rate	Required Detention Volume	Effective C-value
name	sf	#	name	cfs	cf	#
1	20000	0.95	Blue Roof	0.27	2242	0.301

Figure I.8. Output for the Effective C-Value of the Blue Roof

Step 7: Input the properties of the parking lot that will drain into the downstream detention system.

The second upstream area that drains to the downstream detention system is the 20,000 sf parking lot. Since there is no detention system specifically for the parking lot, the effective weighted runoff coefficient remains as 0.85. The workbook will automatically output this value.

UPSTREAM SYSTEM

INPUTS					OUTPUTS	
TDA ID	TDA Area	C-value	Detention System Type	Maximum Release Rate	Required Detention Volume	Effective C-value
name	sf	#	name	cfs	cf	#
1	20000	0.95	Blue Roof	0.27	2242	0.301
2	20000	0.85	None			0.850

Figure I.9. Inputs and Output for the Parking Lot

Step 8: Calculate the release rate to be maintained by the controlled-flow orifice for the downstream detention system. Use the maximum release rate per acre (q) shown in Table 2.9, Chapter 2.

Since the project is 20,000 sf or more, and consists of a multistory office building, this project requires a site connection permit (SCP). In addition, the site is connecting to a 15 in. combined sewer.

Table 2.9. Maximum release rate per acre (cfs/acre) by sewershed type.

q (cfs/acre)	Description
1.0	MS4 areas
0.1	CSS areas

According to Table 2.9, $q = 0.1 \frac{cfs}{acre}$.

$$Q_{DRR} = \frac{q \, * A}{43560} \; or \; 0.046 \;\; [whichever \; is \; greater]$$

where:

Q_{DRR} = maximum release rate for the site (cfs)

q = maximum release rate per acre (cfs/acre) = 0.1 cfs/acre

A = contributing area (sf) = 40,000 sf

$$Q_{DRR} = \frac{0.1 \, \frac{cfs}{acre} * 40{,}000 \; sf}{43560} \, or \; 0.046 \;\; [whichever \; is \; greater]$$

$$Q_{DRR} = 0.092 \; cfs \; > 0.046 \; cfs$$

The maximum release rate is 0.092 cfs.

Step 9: Input the properties of the downstream detention system. Use the maximum release rate from Step 8.

Since the project is 20,000 sf or more, and consists of a multistory office building, this project requires a site connection permit (SCP). The site has a total contributing area of 40,000 sf.

DOWNSTREAM SYSTEM

INPUTS			OUTPUTS	
Permit Type	Total Contributing Area	Maximum Release Rate	Required Detention Volume	Effective C-value
name	sf	cfs	cf	#
CSS - SCP	40000	0.092		

Figure I.10. Inputs for the Downstream Detention System

Step 10: Based on the inputs from Step 9, the workbook will automatically calculate the effective weighted runoff coefficient for the downstream detention system. This calculation is shown below.

$$C_W = \frac{(C_1A_1 + C_2A_2 + \cdots etc.)}{A_t}$$

where:

C_W = weighted runoff coefficient relating peak rate of rainfall and runoff

C_1 = the effective weighted runoff coefficient for the area classified as roof = 0.30

A_1 = the area classified as roof (sf) = 20,000 sf

C_2 = the effective weighted runoff coefficient for the area classified as paved = 0.85

A_2 = the area classified as paved (sf) = 20,000 sf

A_t = contributing area (sf) = 40,000 sf

$$C_W = \frac{(0.30 * 20{,}000\ sf) + (0.85 * 20{,}000\ sf)}{40{,}000\ sf}$$

$C_W = 0.575$

DOWNSTREAM SYSTEM

INPUTS			OUTPUTS	
Permit Type	Total Contributing Area	Maximum Release Rate	Required Detention Volume	Effective C-value
name	sf	cfs	cf	#
CSS - SCP	40000	0.092		0.575

Figure I.11. Output for the Effective C-Value of the Downstream Detention System

Step 11: Identify the rainfall depth (R_D) based on the sewershed type and connection proposal type for the project. Use Table 2.7 in Chapter 2.

Since the project is 20,000 sf or more, and consists of a multistory office building, this project requires a site connection permit (SCP). In addition, the site is connecting to a 15 in. combined sewer.

Table 2.7. Applied rainfall depth by sewershed type and connection proposal type.

R_D	Description
1.85	CSS areas with SCP
1.50	CSS areas with HCP
1.50	MS4 areas wiith SCP
1.10	MS4 areas with HCP

According to Table 2.7, $R_D = 1.85\ in$.

Step 12: Based on the inputs from Step 9, the workbook will automatically calculate the required detention volume through the detention tank. This calculation is shown below.

$$V_V = \frac{R_D}{12} * A * C_W$$

where:

V_V = the maximum required detention volume (or sewer operations volume) (cf)

R_D = rainfall depth (in) = 1.85 in

A = contributing area (sf) = 40,000 sf

C_W = weighted runoff coefficient relating peak rate of rainfall and runoff = 0.575

$$V_V = \frac{1.85\ in}{12} * 40{,}000\ sf * 0.575$$

$$V_V = 3{,}548\ cf$$

DOWNSTREAM SYSTEM

INPUTS			OUTPUTS	
Permit Type	Total Contributing Area	Maximum Release Rate	Required Detention Volume	Effective C-value
name	sf	cfs	cf	#
CSS - SCP	40000	0.092	3548	0.575

Figure I.12. Output for the Required Detention Volume Through the Downstream Detention System

Step 13: Use the controlled-flow orifice equation to determine an appropriate orifice area for the detention tank, by assuming the active storage depth.

In order to minimize the area required for the detention tank, choose the maximum depth that is still feasible according to site limitations and use a re-entrant orifice. In this case, the designer has chosen an active storage depth of 4 ft.

$$Q_O = C_D * A_o * \sqrt{2gH}$$

where:

Q_O = maximum release rate of orifice (cfs) = 0.092 cfs

C_D = coefficient of discharge, 0.52 for re-entrant orifice

A_O = area of orifice (sf)
g = acceleration due to gravity, 32.2 (ft/s^2)
H = maximum hydraulic head above the centerline of the orifice (ft) = 4 ft

$$0.092\ cfs = 0.52 * A_o * \sqrt{2 * 32.2\ \left(\frac{ft}{s^2}\right) * 4\ ft}$$

$A_o = 0.011\ sf$

Step 14: Translate the area of the controlled-flow orifice (A_O) into a diameter and check that it is greater than the minimum diameter of 1 in.

$$A_o = \frac{\left[\pi * \left(\frac{D_o}{2}\right)^2\right]}{144}$$

where:
A_O = area of orifice (sf) = 0.011 sf
D_O = diameter of orifice (in)

$$0.011\ sf = \frac{\left[\pi * \left(\frac{D_O}{2}\right)^2\right]}{144}$$

$D_O = 1.42\ in\ > 1\ in \qquad OK$

Set the orifice diameter to the nearest 0.25-inch interval rounding down, with a minimum orifice diameter of one-inch. In this case, use an orifice diameter of 1.25 inches.

Step 15: Confirm the orifice area of the selected orifice diameter from Step 14.

$$A_o = \frac{\left[\pi * \left(\frac{D_o}{2}\right)^2\right]}{144}$$

where:
A_O = area of orifice (sf)
D_O = diameter of orifice (in) = 1.25 inches

$$A_O = \frac{\left[\pi * \left(\frac{1.25\ in}{2}\right)^2\right]}{144}$$

$A_O = 0.009\ sf$

Step 16: Confirm the required active storage depth in the tank using the orifice area from Step 15.

$$Q_O = C_D * A_o * \sqrt{2gH}$$

where:

Q_O = maximum release rate of orifice (cfs) = 0.092 cfs

C_D = coefficient of discharge, 0.52 for re-entrant orifice

A_O = area of orifice (sf) = 0.009 sf

g = acceleration due to gravity, 32.2 (ft/s^2)

H = maximum hydraulic head above the centerline of the orifice (ft)

$$0.092\ cfs = 0.52 * 0.009\ sf * \sqrt{2 * 32.2\ \left(\frac{ft}{s^2}\right) * H}$$

$$H = 6.0\ ft$$

If the active storage depth is too high, then increase the orifice size by 0.25 inches and re-run Steps 13-14 until a suitable depth is identified. If the active storage depth is too low, then decrease the orifice size by 0.25 inches (but not less than 1 inch) and re-run Steps 13-14. Alternatively, the designer can choose a different orifice configuration as needed to modify the active storage depth.

In this case, the depth is too high to drain via gravity connection to the storm sewer. Using a flush orifice, which has a coefficient of discharge of 0.61, results in an active storage depth of 4.4 ft.

Step 17: Set the dimensions of the detention tank's active storage zone.

Based on the active storage depth of 4.4 ft and the V_V of 3,548 cf, set the interior detention tank dimensions to L: 28.5 ft and W: 28.5 ft. The resulting detention tank has an active storage volume of 3,574 cf. Note that the exterior dimensions of the detention tank will be larger than the dimensions of the active storage zone (28.5'L x 28.5'W x 4.4'D) to accommodate wall thickness, bypass structures, and/or other internal features.

APPENDIX H

Right-of-Way Guidance Materials

- ROW Geotechnical Procedures
- ROW SMP Data Tracking Form

NYC DEPARTMENT OF ENVIRONMENTAL PROTECTION

BUREAU OF ENVIRONMENTAL PLANNING AND ANALYSIS

PROCEDURE GOVERNING

LIMITED GEOTECHNICAL INVESTIGATION

FOR

RIGHT-OF-WAY STORMWATER MANAGEMENT PRACTICES

NYC Stormwater Manual

July 2021

Table of Contents

Attachments:

- Attachment A: Geotechnical Report Summary Table
- Attachment B: Soil Boring Log
- Attachment C: Falling-Head Borehole Test Log
- Attachment D: Soil Sampling Laboratory Test Results (Example)

Limited Geotechnical Investigation

1 General Guidelines

The Limited Geotechnical Investigation consists of:

a) Soil borings to determine the soil characteristics (field observation and laboratory testing) as well as the depths to groundwater table and bedrock where encountered, AND
b) In-situ soil permeability tests to determine infiltration rates of the existing soil.

The minimum required number of soil borings and permeability tests, collectively referred to as B/PTs, is as follows depending on the size (footprint area) of the proposed stormwater management practice (SMP):

- SMPs with areas less than 1000 SF: at least one B/PT per SMP
- SMPs with areas 1000 SF or more and less than 5000 SF: at least two B/PTs per SMP
- Additionally, the Qualified Professional[1] must make a reasonable determination based on the soil textural classifications and the standard penetration tests to determine if additional tests may be needed; this is particularly critical in areas of fill soils where characteristics will vary greatly over small distances.

Where two or more B/PTs are being conducted for a single SMP, the Qualified Professional must select appropriate locations and spacing between the B/PTs to ensure the geotechnical investigation results will be representative of the underlying soil across the footprint of the SMP.

The following sections provide more detail on the soil boring and PT procedures.

1.1 Geotechnical Investigation Locations

Soil borings and permeability tests shall be conducted in separate boreholes no closer than 5 ft apart. If a boulder or other obstruction is encountered during drilling for any SMP, another attempt shall be made within 5 ft - 10 ft of the original borehole. Each borehole should be given a name corresponding to the SMP ID and the test (B/PT) and an accurate coordinate (latitude and longitude) of each borehole should be recorded.

Soil borings and PTs must be performed within the footprint of the SMP. In the event that drilling cannot be conducted within the footprint area, drilling should be done no more than 10 ft beyond the footprint of the SMP.

[1] As defined in Chapter 19.1 of Title 15 of Rules of the City of New York

1.2 Geotechnical Investigation Methodology

1.2.1 Soil Boring Procedure and Equipment

The Qualified Professional shall approve the drilling method that will minimize disturbance to the soil tested from the following list of acceptable equipment:

- Direct Push Method with a 4-inch inner diameter casing
- Hollow-stem auger (HSA) with a 4-inch inner diameter hollow-stem
- Rotary Tri-cone Roller Bit cased by 4-inch inner diameter casing

In the event that no subsurface records (utility records such as water, sewer, etc) were obtainable for drilling and/or the Qualified Professional chooses, pneumatic and/or hand auger is an acceptable method of boring up to the depth of the first soil sample or PT (see **Section 1.2.3**. for soil sampling and PT depths). The reason for conducting this procedure must be properly documented and reported to DEP.

Only water from a hydrant or any clean potable water source shall be used as drilling fluid. It is not acceptable to recycle the drilling fluid or to use drilling mud. Proper sediment control must be used at all times to prevent runoff containing fine or coarse material from entering the catch basins or leaving the work zone.

The Qualified Professional shall be on-site to observe the soil boring operation and keep a continuous and accurate Boring Log for each location recording all pertinent data. Refer to **Section 2.1.1** for details on the Boring Log.

1.2.1.1 Standard Penetration Test

In each soil boring location, a Standard Penetration Test (SPT) shall be conducted continuously in accordance with ASTM D1586 (i.e. a 24-inch long, 2-inch outside diameter split-barrel-sampler driven by blows from a 140-pound hammer falling freely from a height of 30 inches) to the depth detailed in **Section 1.2.3**.

The number of blows required to drive the 24-inch split-barrel sampler every 6-inch increment will be recorded. The Standard Penetration Resistance (N-value) shall be determined as the sum of the blows required to drive the sampler to the second and third 6-inch increments, representing the number of blows per foot.

1.2.1.2 Soil Sampling

The Qualified Professional shall make observations of the soil samples at all depths during drilling and submit observations for each soil boring location as individual Boring Logs.

The Qualified Professional shall collect soil samples that are representative of the actual recovered soil at specific depths for laboratory analysis. Collected samples shall be stored in labeled jars, to be delivered to an approved AASHTO-certified laboratory for subsequent examination and testing. Samples shall be taken and tested as outlined in **Section 1.3**.

1.2.2 Permeability Test Procedure and Equipment

The recommended method for the in-situ permeability test is the falling-head borehole test as outlined below; however, the Qualified Professional may choose to conduct permeability tests following a percolation test or double-ring infiltrometer test procedure, depending on project or site conditions.

Prior to conducting any permeability test, the following conditions shall be checked:

- If a soil boring was conducted within 20 ft. of a planned PT location, the borehole from the soil boring must be completely backfilled before the PT is commenced.
- Clean water must be used in conducting PTs. PTs conducted using “dirty water” creates faulty results, which shall be rejected, and retest will be required.
- Permeability tests shall not be performed when the ambient temperature is below 0°C, in frozen soils, or with water at temperatures less than 5°C (see **Section 1.2.2.4** on temperature measurement requirements).

1.2.2.1 Falling-head borehole test procedure

The falling-head borehole test procedure is as follows:

- Drive the 4-inch inner diameter casing to the required test depth (refer to soil boring procedure for allowable equipment). The space (annulus) between the casing and borehole must be kept at a minimum. If the casing cannot be driven and a larger hole is first bored to allow for the casing, the annulus must be backfilled and packed with drill cuttings before any water is introduced for testing into the casing.
- Measure the depth to the bottom of the hole to the nearest inch.
- Ensure that the depth to the bottom of the hole is within 1 inch of the depth to the bottom of the casing.
- Place approximately 6 - 8 inches of coarse sand (4.75mm – 2mm) at the bottom of the casing.
- Wash out casing using a continuous flow of clean water at low water pressure (the water shall not disturb the coarse sand layer at the bottom of the casing) until the water exiting the casing runs clear with no discoloration.
- Saturate the soil beneath the bottom of the casing for at least thirty (30) minutes using clean water.
- Fill casing to the top with clean water and record the temperature of the water at the bottom of the casing at the start of the test (see **Section 1.2.2.4** for details on temperature measurement).
- Record the time at the beginning of the test.
- Record the falling water level in the casing at 1, 2, 3, 4, 5, 10, and 15 minutes after the beginning of the test or until the water level in the casing has stopped falling.

- At the conclusion of the test, fill the casing to the top with clean water and maintain the water at this level for five (5) minutes.
- Repeat the test once for each testing depth using the same procedure.

Falling-head borehole tests may be terminated after the 30-minute saturation period and reported accordingly for the following conditions:

- If the casing is completely filled during the saturation period and there is no visible drop in water level after 30 minutes, the falling-head borehole test shall be reattempted for the same depth at another location between 5 ft to 10 ft away. If there is no visible drop in water level after 30 minutes at the reattempted location, the falling-head borehole test shall be terminated for that depth only and the soil permeability rate shall be reported as “0.000 in/hr”.
- If the casing cannot be filled due to rapid infiltration (RI) during the saturation period and no water is retained in the casing after 30 minutes, the falling-head borehole test shall be reattempted for the same depth at another location between 5 ft to 10 ft away. If rapid infiltration is observed during the saturation period for the reattempt, the falling-head borehole test shall be terminated for that depth only and the permeability coefficient reported as “RI”.

The Qualified Professional must log continuous data during this test and report them accurately in Falling-head Borehole Test Logs (FH Logs). Refer to the below and **Section 2.1.3** for details on the PT Log.

Average permeability rates shall be calculated based on a modification of ASTM D6391 using the following formula. The FH Log template with the formula and associated calculation methods is provided. In general, no permeability calculations are necessary at the time of drilling since permeability values (and other variables used to calculate permeability values) are automatically calculated in the FH Log once all the data recorded during the falling-head borehole test are inputted into the template.

$$K_m = \pi \cdot R_t \cdot \frac{D \cdot \left(\ln \frac{h_1}{h_2}\right)}{11 \cdot (t_2 - t_1)}$$

$$R_t = \frac{2.2902(0.9842^{\mathrm{T}})}{\mathrm{T}^{0.1702}}$$

Where:	K_m	= Mean permeability [in/hr], and $K_m = \sqrt{k_h \cdot k_v}$
	k_h	= Horizontal permeability [in/hr]
	k_v	= Vertical permeability [in/hr]
	D	= Inner diameter of casing [in]
	h	= Height of water above bottom of casing at time t [in]
	t	= Time [hr]

R_t = Ratio of viscosity of water at test temperature to the viscosity of water at 20 °C

T = temperature [°C]

1.2.2.2 Percolation test procedure

Percolation tests are commonly used for on-site sewage (septic) and stormwater (dry well) systems. They differ from cased borehole tests in that there is no casing and there is no control for water lost at the sides of the test pit hole during percolation testing. The percolation test method shall not be utilized for proposed SMP locations less than 10 feet from buildings or underground structures. Percolation tests must be conducted in accordance with the NYS procedure[2] for onsite sewage treatment systems.

Following the above percolation test procedure will result in a measurement of the stabilized rate of percolation of the soil. This stabilized percolation rate must be translated to a permeability, or infiltration rate, using a reduction factor that accounts for water lost at the sides of the test pit. The following equation may be used to calculate the infiltration rate:

$$I = \frac{P_s}{R_f}$$

where:

I: infiltration rate (in/hr)

P_s: stabilized rate of percolation (in/hr)

R_f: Reduction factor of 1.92

The reduction factor assumes the percolation rate is affected by the depth of water in the test hole and that the percolating surface of the hole is in uniform soil. If there are site conditions that cause significant deviations from either assumption, such as noticeably different soil strata along the percolation test hole, then this methodology is not appropriate for determining infiltration rates.

1.2.2.3 Double-Ring Infiltrometer test procedure

Double-Ring Infiltrometer tests require less equipment compared to the other permeability test procedures but can be more difficult to use in very pervious or very impervious soils, in dry or stiff soils, or if the rings are fractured when installed. Double-Ring Infiltrometer tests shall be conducted in accordance with the latest version of ASTM D3385, the Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer.

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[2] Full procedure available at the following links (accessible as of July 2021): https://www.dec.ny.gov/docs/water_pdf/2014designstd.pdf or https://www.health.ny.gov/environmental/water/drinking/wastewater_treatment_systems/docs/design_handbook.pdf

1.2.2.4 Temperature Measurement

Temperatures shall be measured in °C using equipment meeting the specifications as shown in Table 1 and calibrated against a National Institute of Standards and Technology (NIST) Standard or with certified calibration traceable to NIST.

Table 1 – Acceptable Temperature Measurement Equipment

Equipment	Specifications
Liquid-in-glass thermometer (nonmercury)	• Temperature range, at least -5 to +45°C • 0.5°C gradations or smaller • Calibrated accuracy within 1 percent of full scale or 0.5°C, whichever is less
Thermistor	• Calibrated accuracy within 0.1 to 0.2°C • Digital readout to at least 0.1°C

1.2.3 Geotechnical Investigation Depths

The minimum depth for all soil borings is 20 ft or 5 ft below the SMP base (i.e. the depth of the infiltrating surface), whichever is deeper.

Bulk soil samples for laboratory testing shall be collected and analyzed for every 2 ft of soil depth, starting at the 3-5 ft depth then every 2 ft interval thereafter to the extent possible to the full soil boring depth. If different soil stratums are encountered within an interval, the Qualified Professional is recommended to recover separate samples for each stratum.

PTs must be conducted at the depth of the SMP base. Qualified Professionals are recommended to conduct additional PTs at depths beyond the SMP base if soils with high fines are observed at the shallow depths and sandy soils are observed at deeper depths, which may allow for the use of stone columns for infiltration.

For example, a SMP that infiltrates at 5 ft depth requires, at a minimum:

- Soil boring to 20 ft
- Soil samples collected and analyzed at the following depths: 3-5 ft, 5-7 ft, 7-9 ft, 9-11 ft, 11-13 ft, 13-15 ft, 15-17 ft, 17-19 ft
- PT at 5 ft

Qualified Professionals should take into account any proposed surface elevation changes when determining appropriate geotechnical investigation depths.

1.3 Geotechnical Laboratory Testing

Laboratory tests shall be conducted by an AASHTO-certified laboratory to determine the distribution of particle sizes of the soil – particularly the fines (silts and clays) content – in accordance with ASTM D422.

2 Geotechnical Report

2.1 Geotechnical Investigation Data

Geotechnical reports must include a Geotechnical Report Summary Table, detailed boring logs, and permeability test logs. Additionally, field-measured B/PT locations must be accurately recorded and submitted on a map that also shows the location of the SMP(s).

2.1.1 Geotechnical Report Summary Table

Pertinent data from the soil borings, PTs, laboratory test results, and any other information acquired during the geotechnical investigation shall be summarized in the Geotechnical Report Summary Table format provided (see Attachment A).

2.1.2 Boring Logs

Separate boring logs must be prepared for all soil borings. An example boring log template is provided as Attachment B. At a minimum, boring logs must include the information listed below:

- Identification number (ID No.)
- Soil boring location and coordinates (latitude/longitude)
- Description of equipment (drilling, SPT, soil sampling, etc)
- Weather
- Number of blows per 6-inch intervals of continuous penetration
- Length of sample recovery (inches) for each 2-ft interval
- Depths of soil samples retrieved for laboratory analysis
- Thickness of each soil stratum encountered (including pavement, fill or topsoil layers).
- Characteristics of the soil (based on field observations) for all depths, including:
    1. Soil description per Modified Burmister
    2. Soil classification per Unified Soil Classification System (USCS), in parentheses
    3. Color
    4. Soil moisture (dry, moist, or wet)
    5. Soil consistency:
        a. for Cohesive soil: very soft, soft, medium stiff, stiff, very stiff, hard
        b. for Granular soils: very loose, loose, medium dense, dense, very dense
    6. If present:
        a. Debris (brick, concrete, wood, glass, etc.)
        b. Cobbles, boulders, etc.
        c. Odor (organic, chemical, etc.)
        d. Notable soil formations which may affect permeability (e.g. "bull's liver", glacial till, etc.)
        e. Indication of possible contamination (ash, petroleum, slag, etc.)
        f. Decomposed vegetation
- Depth to groundwater and/or bedrock, if encountered
- Other subsurface conditions encountered during drilling (e.g. utilities, structures, etc.)

- Additional observations noted during soil boring

2.1.3 Permeability Test Logs

Permeability test logs must be submitted for all PTs, including those that were terminated. At a minimum, PT logs must include the following:

- Permeability test method
- PT ID number
- Weather and ambient temperature
- PT location and coordinates (latitude/longitude)
- Description of equipment utilized
- PT depth
- Depth to groundwater and/or bedrock, if encountered
- Water temperature at the start of the test
- All water depth readings as required by test procedure
- Calculation steps
- Resulting permeability rates

Falling-head borehole test results shall be reported on the FH Log (see Attachment C). The following are additional notes for reporting on falling-head borehole test results:

- Early termination of falling-head borehole tests shall be noted in the "Inspectors Remarks" section of the FH Logs and in the Geotechnical Report Summary Table under "General Geotech Notes". No field data shall be reported as "Depth (in)", and no permeability values shall be calculated for terminated falling-head tests.
- The FH Log template contains default time values of 1, 2, 3, 4, 5, 10, and 15 minutes after the start of the test. If the water level drops below the casing before the 15-minute measurement period, these default values must be modified to the actual time values for which water depth measurements were recorded.
- If the permeability rate cannot be calculated (for example, due to RI), the FH Log shall clearly indicate that calculations are not valid.

2.1.4 Laboratory Test Results

Laboratory testing and reporting must include a sieve analysis of soil samples and plotting of gradation curves, as well as soil classification based on the USCS.

The following USCS-classified sieve sizes are to be included with data points for all sampled depths overlaid on the same gradation curve:

4"
3"
1-1/2”
3/4"
3/8”
#4
#10

#20
#40
#60
#100
#200

An example of an acceptable format for reporting soil sieve analyses and gradation curves is provided as Attachment D.

NYC Department of Environmental Protection

Project: [Project Description]
Prepared By: [Consultant/Sub Name]

Geotechnical Report Summary Table

Environmental Protection

SMP ID No.	Soil Laboratory Results				Permeability Analysis				Groundwater Table Depth (ft)	Bedrock Depth (ft)	General Geotechnical Notes	Additional Notes
	Boring ID No.	Depth (ft)	USCS Symbol	% Passing No 200 Sieve	Permeability Test ID No.	PT Method	Permeability Test Depth (ft)	Average Permeability Rate (in/hr)				

Notes:

Only numbers should be inputted in the '% Passing No 200 Sieve', 'Average Permeability Coef. (ft)', 'Groundwater Table Depth (ft)', and 'Bedrock Depth (ft)' columns.
For the '% Passing No. 200 Sieve' column, values must be between 0 and 1. (i.e. use either 0.15 or 15% not 15). Numbers greater than 1 will not be accepted.
Please refer below for allowable exceptions and other specific instructions:
(NE = not encountered, NR = no record, NP = not performed)

Column	Exception(s)
'USCS Symbol', '% Passing No 200 Sieve'	If soil sampling was cancelled due to groundwater, bedrock, obstructions, etc., enter "NP" (details should be included in the 'General Geotech Notes' column) If soil sample could not be obtained or recovery was too low to be analyzed, enter "NR"
Average Permeability Coef. (ft)	For high permeabilities where the water level drop rate could not be measured, enter "RI" If a PT could not be conducted at specific depths, input depth with "NP" as the Permeability Rate (details should be included in the 'General Geotech Notes' or 'Additional Notes' as applicable)
Groundwater Table Depth (ft)	Enter the depth that groundwater was encountered. If groundwater was not encountered, enter "NE" If perched water was encountered, enter "NE" (but include in the 'General Geotech Notes' column)
Bedrock Depth (ft)	Enter the depth that bedrock was encountered. If bedrock was not encountered, enter "NE"

Relevant information to include under General Geotechnical Notes include (but not limited to): refusal (please provide possible cause of refusal), suspected contamination, perched water, etc.

Environmental Protection	COMPANY NAME/LOGO	Boring ID No. <XXXX>

Project: <Project Name/Description>	Location: <Description of Location>

INSPECTOR:	<name>	DRILLER:	<name>	Start Date:	<date>	Weather:	<weather>
CONTRACTOR:	<name>	HELPER:	<name>	Start Time:	<time>		
OVERSIGHT:	<name>						

Total Boring Depth:	<##> ft	Drill Bit Type:	<type>		Weight of Hammer for casing:	<##>	lbs
Rig Type:	<type>	Casing Inner Diameter:	4	in	Weight of Hammer for spoon:	<##>	lbs
		Depth of Casing:		ft	Type of Hammer:	<type>	
Depth to Groundwater Table (bgs):	<##> ft				Drop:		in
Depth to Bedrock (bgs):	<##> ft				Split Spoon Diameter:		in

<XXXX> BORING LOG

Depth Below Ground Surface (ft)	Soil Sample Retrieval and Sample No.	Soil Description (Field Observations)	SPT Blows per 6"	N Value	Recovery Length (inches)	Remarks
0		asphalt pavement				
5						
10						
15						
20						

Boring terminated at xx feet below ground surface

Latitude: <latitude> Longitude: <longitude>

Inspector's Remarks:

Falling-Head Borehole Test Log

COMPANY NAME/LOGO		PT ID No.	<ID>		
		Sheet	<#>	of	<#>

Project: <Project Name/Description>		LOCATION:	

INSPECTOR:	<name>	DRILLER:	<name>	Start Date:	<date>	Weather:	<weather and ambient temperature>
CONTRACTOR:	<name>	HELPER:	<name>	Start Time:	<time>		
OVERSIGHT:	<name>						

Depth of PT:	<depth>	ft	Drill Bit Type:	<type>		Weight of Hammer for casing:	140	lbs
Rig Type:	<type>		Casing Internal Diameter:	4	in	Type of Hammer:	<type>	
			Casing Length:	<length>	in			

ASTM D-6391
PERMEABILITY COEFFICIENT (Km) FORMULA:

General Formula:

$$K_m = \pi R_t \times \frac{\left[D\left\{Ln\left(\frac{h_1}{h_2}\right)\right\}\right]}{11 \times (t_2 - t_1)}$$

Formula for 4" internal diameter casing (in/hr):

$$K_m = 1.142 R_t \times \frac{\left[Ln\left(\frac{h_1}{h_2}\right)\right]}{(t_2 - t_1)}$$

where: $R_t = {2.2902(0.9842^T)}/{T^{0.1702}}$

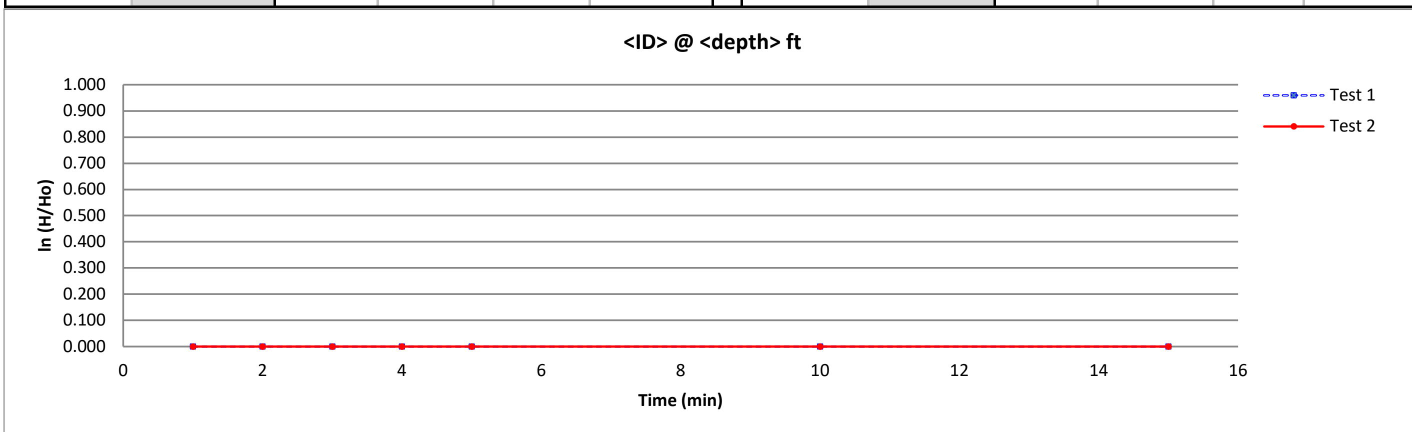
<ID> @ <depth> ft

TEST 1

Water temperature (°C), T:			Rt=	-	
FIELD DATA		CALCULATED DATA			
Time (min)	Depth (in)	Height (in)	Ln (H/Ho)	$(t_1$-$t_2)$	*Kv (in/hr)
1		-	-	0.017	-
2		-	-	0.017	-
3		-	-	0.017	-
4		-	-	0.017	-
5		-	-	0.017	-
10		-	-	0.083	-
15		-	-	0.083	-

TEST 2

Water temperature (°C), T:			Rt=	-	
FIELD DATA		CALCULATED DATA			
Time (min)	Depth (in)	Height (in)	Ln (H/Ho)	$(t_1$-$t_2)$	*Kv (in/hr)
1		-	-	0.017	-
2		-	-	0.017	-
3		-	-	0.017	-
4		-	-	0.017	-
5		-	-	0.017	-
10		-	-	0.083	-
15		-	-	0.083	-



TEST 1 FINAL RESULTS		
Time Weighted Average Permeability Coefficient	K_m=	0.0000 in/hr

TEST 2 FINAL RESULTS		
Time Weighted Average Permeability Coefficient	K_m=	0.0000 in/hr

AVERAGE <ID> @ <depth> ft			
Time Weighted Average Permeability Coefficient	*K_m=	0.0000	in/hr

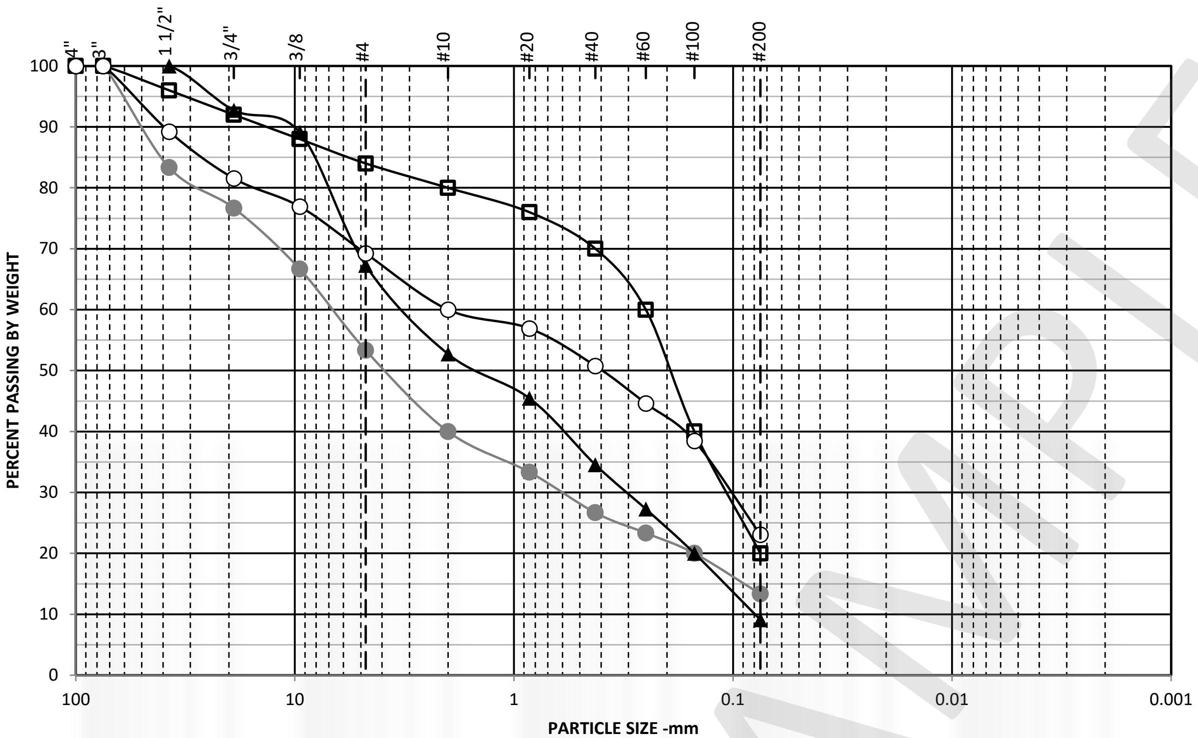
Coordinates:	
Longitude:	<longitude>
Latitude:	<latitude>

Inspectors Remarks:

DEFINITION OF VARIABLES

*K_m= Mean permeability rate
T = Temperature of permeant (water), in °C
Ln = Natural Logarithmic
t_1 = Time at the start of the test in the same units selected for Km
R_t = Ratio of viscosity of water at test temperature to the viscositye of water at 20°C
t_2= Time at the end of the test in the units selected for Km
h_1= Height of the water above the bottom of the casing at the start of the test in the same units selected for Km
h_2= Height of the water above the bottom of the casing at the end of the test in the same units selected for Km

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	



Boring ID No. XXXX

Symbol	●	□	▲	○	◆
Depth	5-7	7-9	11-13	18-20	-
% Gravel	46.67	16.00	32.73	30.77	-
% Sand	40.00	64.00	58.18	46.15	-
% Fines	13.33	20.00	9.09	23.08	-
% -2µ	-	-	-	-	-
Cc	-	-	0.36	-	-
Cu	-	-	41.54	-	-
D_{100} (mm)	75.00	75.00	37.50	75.00	-
D_{60} (mm)	7.13	0.25	3.38	2.00	-
D_{30} (mm)	0.64	0.11	0.32	0.11	-
D_{10} (mm)	-	-	0.08	-	-
USCS	SP-SM	SM	SP	SM-ML	-
w (%)	-	-	-	-	-

Particle Size (Sieve #)	**Percent Finer**				
	●	□	▲	○	◆
4"	100.0	100.0	100.0	100.0	-
3"	100.0	100.0	100.0	100.0	-
1 1/2"	83.3	96.0	100.0	89.2	-
3/4"	76.7	92.0	92.7	81.5	-
3/8"	66.7	88.0	89.1	76.9	-
4	53.3	84.0	67.3	69.2	-
10	40.0	80.0	52.7	60.0	-
20	33.3	76.0	45.5	56.9	-
40	26.7	70.0	34.5	50.8	-
60	23.3	60.0	27.3	44.6	-
100	20.0	40.0	20.0	38.5	-
200	13.3	20.0	9.1	23.1	-

Depth (ft bgs)	Symbol	DESCRIPTION AND REMARKS
5-7	●	sandy silt
		Color/Odor/Impurities: brown
7-9	□	silt and sand
		Color/Odor/Impurities: dark brown - brown
11-13	▲	poorly graded sand, little silt
		Color/Odor/Impurities: light brown
18-20	○	silt and clay, some sand
		Color/Odor/Impurities: brown/gray
-	◆	-

PARTICLE SIZE DISTRIBUTION

Project: <Project Description>

Company Name/Logo

Laboratory Name

Comments:

ROW SMP Data Tracking Form	Updated July 2021

PROJECT LEVEL DATA

No.	Field Name	Field Description	Response
1	Project ID	Contract ID of project	
2	Project Description	Short description of project	
3	Project Borough	Borough (Bronx, Manhattan, Queens, Brooklyn, or Staten Island)	
4	Project Area	Approximate area of entire project in **acres**	
5	Agency	City agency (DDC, DOT, etc) managing project	
6	Contact First Name	Project manager first name	
7	Contact Last Name	Project manager last name	
8	Design Completion Date	Design completion date, may use date when contract drawings were finalized	
9	Construction NTP Date	Enter date Notice to Proceed (NTP) was issued for construction	
10	Construction Guarantee End Date	Enter date when the contractor guarantee period for SMPs ended	
11	Construction Project Acceptance Date	Enter date when all SMPs were accepted	
12	Construction End Date	Enter construction contract end date	

ROW SMP Data Tracking Form	Updated July 2021

SMP INFORMATION - SEPARATE REPONSES ARE REQUIRED FOR EACH SMP, ADD ADDITIONAL COLUMNS FOR EACH SMP

No.	Field Name	Field Description	Response <SMP1>
1	SMP ID	Assign a unique ID to each SMP. Alphanumeric only.	
2	SMP Type	Choose from: ROW Precast Porous Concrete, ROW Bioswale with Type D inlet, ROW Infiltration Basin, ROW Bioswale. *If ROW Infiltration Basin, indicate if grass or concrete.	
3	SMP X-Coordinate	X-coordinate of SMP using NAD 1983 State Plane Long Island FIPS 3104 Feet, measured at asset inlet or upstream corner for Porous Concrete	
4	SMP Y-Coordinate	Y-coordinate of SMP using NAD 1983 State Plane Long Island FIPS 3104 Feet, measured at asset inlet or upstream corner for Porous Concrete	
5	SMP BBL	BBL nearest to SMP	
6	Disturbed impervious drainage area of SMP	Disturbed drainage area (in SF) specific to the SMP, as delineated in Chapter 6 - SMP Sizing	
7	SMP storage volume	Storage volume of the SMP as calculated according to Chapter 4	
8	SMP Length	Length of SMP as measured parallel to the curb, in ft	
9	SMP Width	Width of SMP as measured perpendicular to the curb, in ft	
10	SMP Features	Indicate if SMP utilizes stormwater chambers and/or stone columns	
11	SMP Standards Date	Indicate date of ROW GI Standards referenced for project	
12	Soil Depth	For ROW Bioswale (with or without Type D inlet), indicate depth of soil layer - 1.5 ft or 2 ft. Put zero for other SMP Types.	
13	Stone Depth	Indicate depth of stone layer in ft	
14	Volume of Stormwater Chamber	If SMP utilizes stormwater chamber, volume of stormwater chamber in CF. Leave blank if no stormwater chamber.	
15	Number of Stone Columns	If SMP utilizes stone columns, number of stone columns	
16	Depth of Stone Columns	Indicate number of feet below ground surface the stone columns extend to	
17	HDPE Barrier?	Indicate 'Y' or 'N' if SMP utilizes HDPE barrier	
18	Bridge Connection?	Indicate 'Y' or 'N' if SMP is hydraulically connected to another SMP	
19	Curb Type	Choose from: Bluestone, Bluestone Cobblestone, Cobblestone, Concrete, Granite, Granite Cobblestone, None, or Steel-Faced	
20	Tree Latin Name and Cultivar	If SMP has a tree, indicate latin name and cultivar of tree. Otherwise leave blank.	
21	Planting Plan (Wet or Dry)	If SMP is ROW Bioswale (with or without Type D inlet), indicate whether planting plan is for wet sites, dry sites, or combination	
22	Planting Plan (Sun or Shade)	If SMP is ROW Bioswale (with or without Type D inlet), indicate whether planting plan is for sunny, shady, or mixed	
23	Planting Plan (Residential or Industrial)	If SMP is ROW Bioswale (with or without Type D inlet), indicate whether planting plan is for residential or commercial/industrial	
24	Soil Boring ID	ID of soil boring conducted for the SMP	
25	Soil Boring X-Coordinate	x-coordinate of soil boring using NAD 1983 State Plane Long Island FIPS 3104 Feet	

26	Soil Boring Y-Coordinate	y-coordinate of soil boring using NAD 1983 State Plane Long Island FIPS 3104 Feet
27	Soil Sample Depth at SMP base	Depth of soil sample taken at the SMP base. For ROW Porous Concrete, should be 3'-5' and for all others should be 5'-7'
28	Soil Sample USCS at SMP base	Based on lab analysis, USCS Soil Classification symbols for soil sample taken at SMP base
29	Soil Sample % fines at SMP base	Based on lab analysis, % passing No. 200 sieve for soil sample taken at SMP base
30	Stone Column Soil Sample Depth	Depth of soil sample for which the stone column penetrates into. Leave blank if no stone columns.
31	Stone Column Soil Sample USCS	Based on lab analysis, USCS Soil Classification symbols for soil sample at stone column depth. Leave blank if no stone columns.
32	Stone Column Soil Sample % fines	Based on lab analysis, % passing No. 200 sieve for soil sample at stone column depth. Leave blank if no stone columns.
33	Soil Boring - Groundwater	Enter depth (ft) of groundwater encountered during soil boring. Put "NE" if not encountered.
34	Soil Boring - Bedrock	Enter depth (ft) of bedrock encountered during soil boring. Put "NE" if not encountered.
35	Permeability Test ID	ID of permeability test conducted for the SMP
36	Permeability Test X-Coordinate	x-coordinate of permeability test using NAD 1983 State Plane Long Island FIPS 3104 Feet
37	Permeability Test Y-Coordinate	y-coordinate of permeability test using NAD 1983 State Plane Long Island FIPS 3104 Feet
38	Permeability Test Depth	Depth below ground surface of permeability test taken to represent SMP base, in ft
39	Permeability Test Method	Indicate which permeability test procedure was utilized (falling-head borehole, percolation, or double-ring infiltrometer)
40	Permeability Test Result	Result of permeability test at SMP base, in inches per hour