NOTICE OF ADOPTION OF FINAL RULE

NEW YORK CITY DEPARTMENT OF ENVIRONMENTAL PROTECTION

NOTICE IS HEREBY GIVEN PURSUANT TO THE AUTHORITY VESTED IN THE COMMISSIONER OF THE DEPARTMENT OF ENVIRONMENTAL PROTECTION by Section 1043(a) of the New York City Charter (City Charter) and Section 24-553 of the Administrative Code of the City of New York (Administrative Code), that the Department of Environmental Protection is amending its rules governing management of construction and post-construction stormwater sources (Title 15, chapter 19.1 of the Rules of the City of New York ("RCNY")).

Statement of Basis and Purpose

The New York City Department of Environmental Protection ("DEP" or "Department") is amending its rules governing management of construction and post-construction stormwater sources (Title 15, chapter 19.1 of the Rules of the City of New York ("RCNY")).

Section 1403(b-1) of the Charter of the City of New York provides that the Commissioner of Environmental Protection ("Commissioner") has "the power to administer and enforce provisions of law, rules and regulations relating to the management and control of discharges and runoff from public and private property, including but not limited to stormwater discharges, which may convey pollutants and other materials that may enter and have an adverse impact on the waters of the state." Title 24 of the Administrative Code of the City of New York, Chapter 5-A establishes stormwater management controls for construction projects to reduce the flow of stormwater runoff and water borne pollutants into sewers that empty directly into the waters of the state or that overflow into such waters because of rain or snowmelt that exceeds the design capacity of wastewater treatment plants.

The amendments to Chapter 19.1 would revise several Appendices to the NYC Stormwater Manual, which provide additional procedural and technical guidance to owners, developers and applicants.

The amendments are minor corrections or clarifications made in Appendix D *Stormwater Management Practice Sizing Examples*, Appendix E *Site Design Example*, Appendix F *Controlled Flow Pump Workbook*, and Appendix G *Detention in Series Examples.*

A public hearing regarding the rule was held on July 1, 2024. As there were no public comments received pursuant to that hearing, no revisions have been made to the final rule.

New material is <u>underlined</u>. Deleted material is shown in [brackets].

"Shall" and "must" denote mandatory requirements and may be used interchangeably in the rules of the department, unless otherwise specified or unless the context clearly indicates otherwise.

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:

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

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

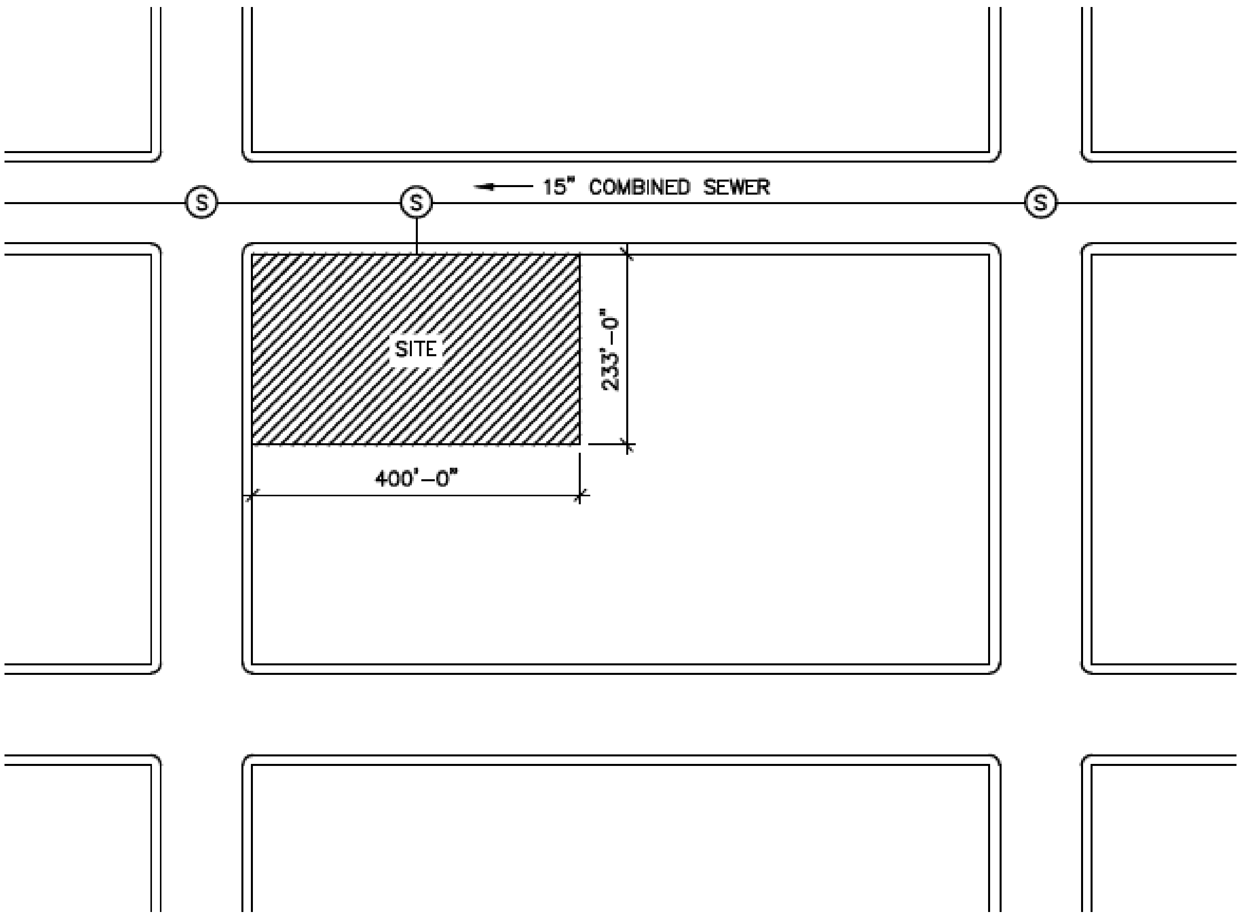


Figure [F]D.1. Schematic of Site (Not to Scale)

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.

Compute the maximum storage depth in ft. of a detention facility with a Re-entrant orifice tube outlet, SDR, with a CD of 0.52, by the equation:

$$S_{DR} = 1{,}930\ (Q_{DRR})^2/(d_O)^4 + d_O/24$$

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)]

S_{DR} = the maximum storage depth in ft. for a Re-entrant orifice tube outlet

Q_{DRR} = the detention facility maximum release rate in cfs

d_O = the nominal dia. of the orifice tube outlet in in.

$$S_{DR} = 1{,}930\ (0.214)^2/(2)^4 + 2/24$$

H = 5.6 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] 5.6 ft and the V_V of 11,035 cfs, set the interior detention tank dimensions to L: [45.5] 44.5 ft and W: [45.5] 44.5 ft. The resulting detention tank has an active storage volume of [11,179] 11,089 cf. Note that the exterior dimensions of the detention tank will be larger than the dimensions of the active storage zone ([45.5] 44.5'L x [45.5] 44.5'W x [5.4] 5.6'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

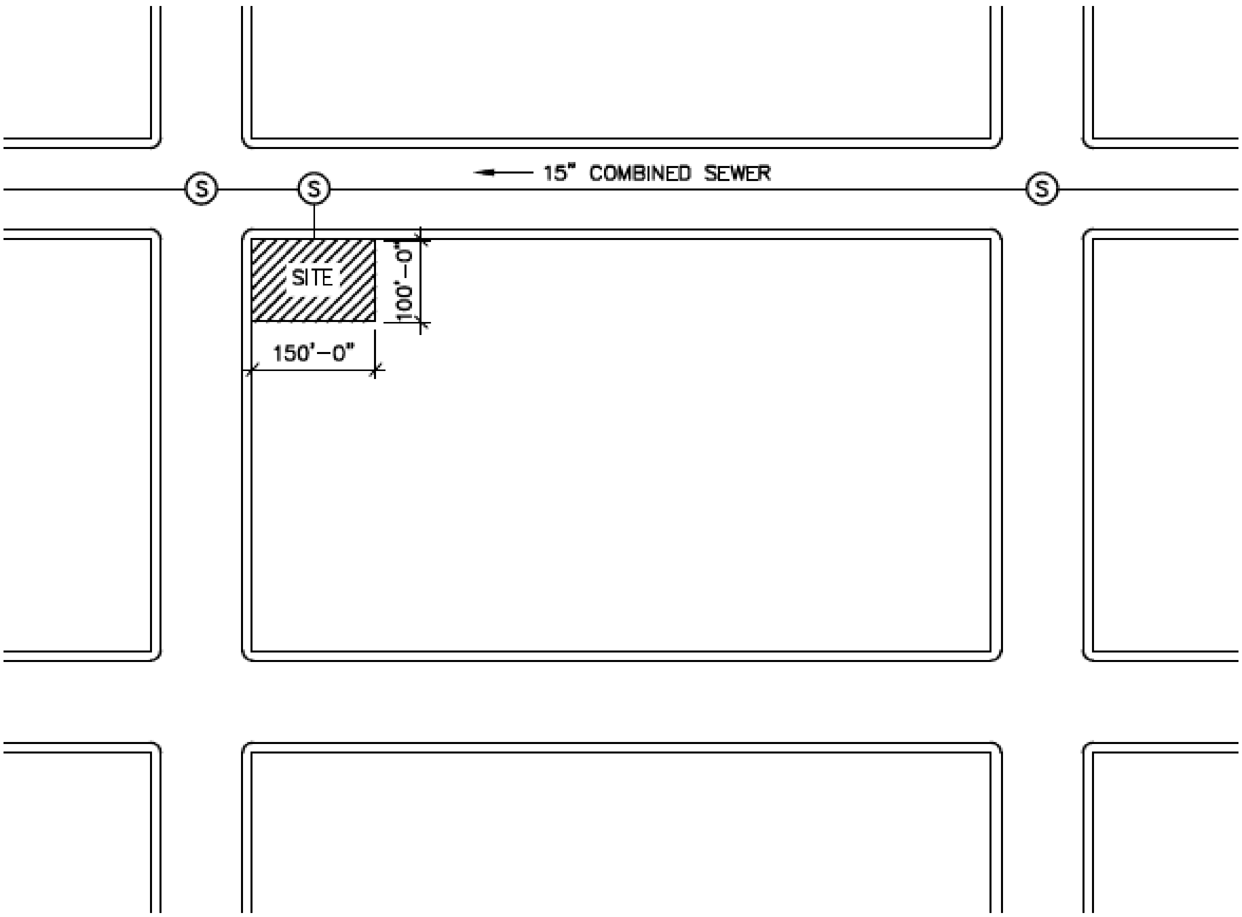


Figure [F]D.2. Schematic of Site (Not to Scale)

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.

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

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) = 2,000 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.

Compute the maximum storage depth in ft. of a detention facility with a Re-entrant orifice tube outlet, SDR, with a CD of 0.52, by the equation:

$$S_{DR} = 1{,}930\ (Q_{DRR})^2/(d_O)^4 + d_O/24$$

where:

S_{DR} = the maximum storage depth in ft. for a Re-entrant orifice tube outlet
Q_{DRR} = the detention facility maximum release rate in cfs
d_O = the nominal dia. of the orifice tube outlet in in.

$$S_{DR} = 1{,}930\ (0.046)^2/(1)^4 + 1/24$$

H = [5.6 ft] 4.13 ft

[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)]

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] 4.13 ft and the V_V of 1,131 cf, set the interior detention tank dimensions to L: [15.5] 6.6 ft and W: [15.5] 6.6 ft. The resulting detention tank has an active storage volume of [1,177] 1,138 cf. Note that the exterior dimensions of the detention tank will be larger than the dimensions of the active storage zone ([15.5]16.6'L x [15.5]16.6'W x [4.9]4.13'D) to accommodate wall thickness, bypass structures, and/or other internal features.

Detention Tank - MS4 with SCP

A 25,050sf 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

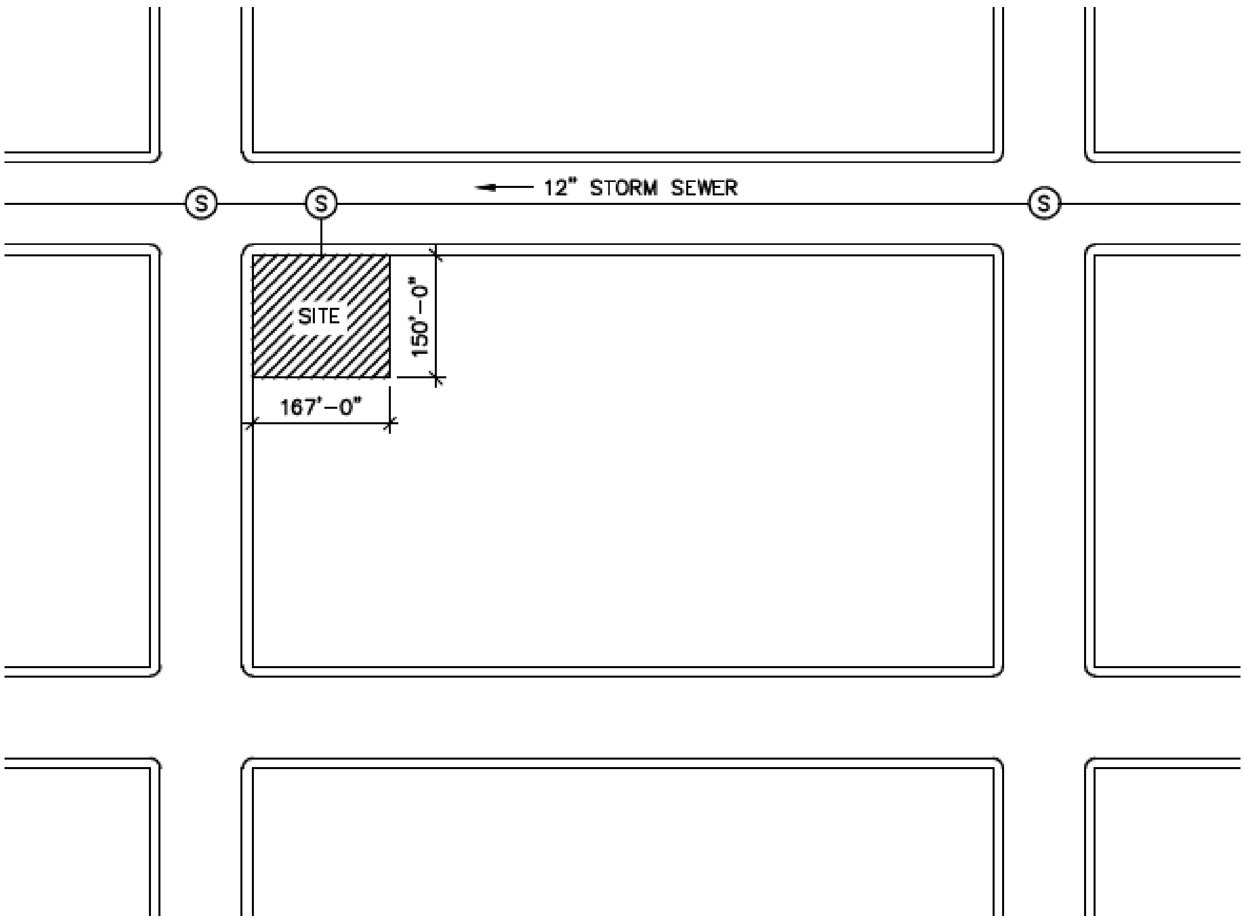


Figure [F]D.3. Schematic of Site (Not to Scale)

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.

[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)]

Compute the maximum storage depth in ft. of a detention facility with a Re-entrant orifice tube outlet, SDR, with a CD of 0.52, by the equation:

$$S_{DR} = 1{,}930\ (Q_{DRR})^2/(d_O)^4 + d_O/24$$

where:

-
S_{DR} = the maximum storage depth in ft. for a Re-entrant orifice tube outlet
Q_{DRR} = the detention facility maximum release rate in cfs
d_O = the nominal dia. of the orifice tube outlet in in.

$$S_{DR} = 1{,}930\ (0.575)^2/(3.5)^4 + 3.5/24$$

H= 4.4 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 [4.2]4.4 ft and the V_V of 2,621 cf, set the interior detention tank dimensions to L: [25]24.5 ft and W: [25]24.5 ft. The resulting detention tank has an active storage volume of [2,625]2,641 cf. Note that the exterior dimensions of the detention tank will be larger than the dimensions of the active storage zone ([25]24.5'L x [25]24.5'W x [4.2]4.4'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

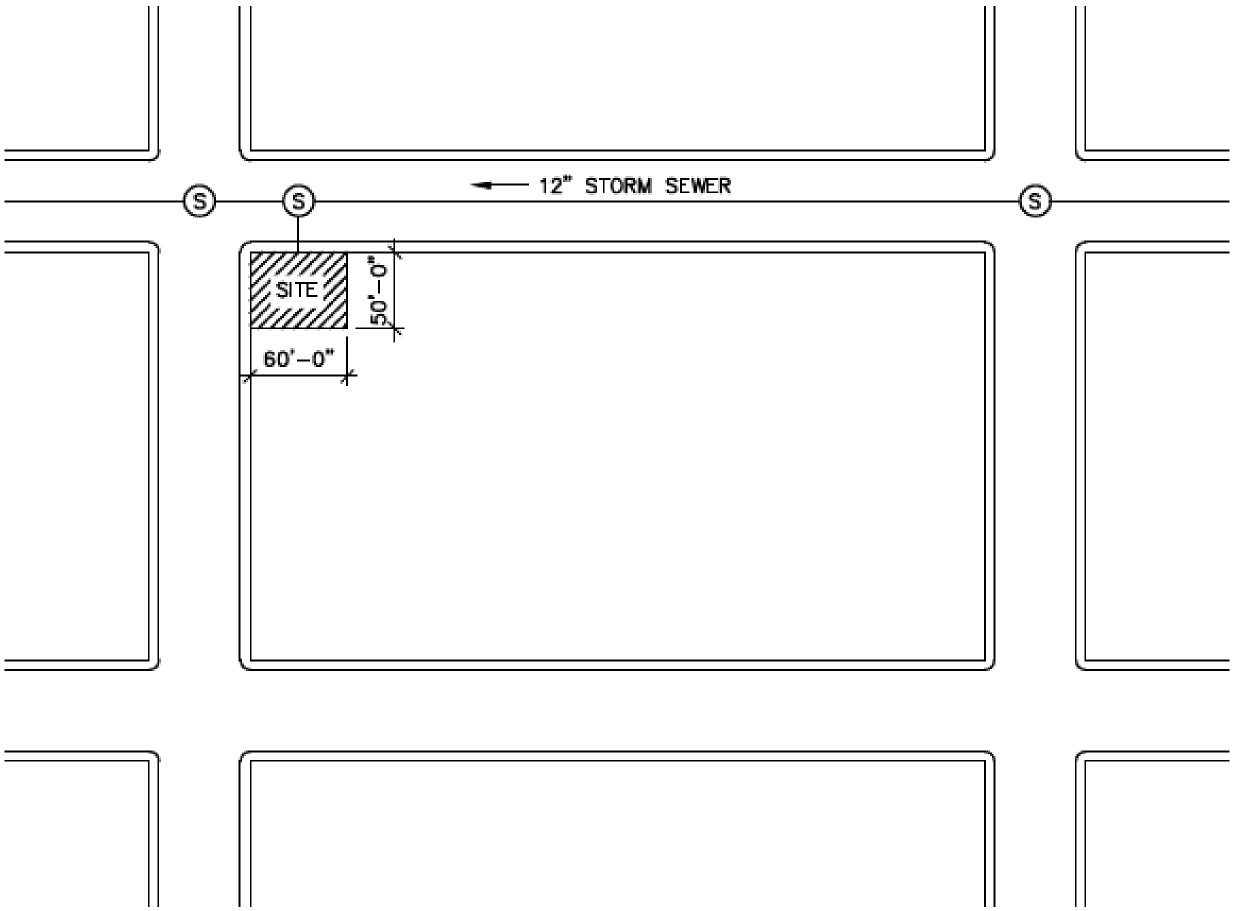


Figure [F]D.4. Schematic of Site (Not to Scale)

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.

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

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) = 2,100 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.

$$S_{DR} = 1{,}930\ (Q_{DRR})^2/(d_O)^4 + d_O/24$$

Compute the maximum storage depth in ft. of a detention facility with a Re-entrant orifice tube outlet, SDR, with a CD of 0.52, by the equation:

$$S_{DR} = 1{,}930\ (Q_{DRR})^2/(d_O)^4 + d_O/24$$

where:

SDR = the maximum storage depth in ft. for a Re-entrant orifice tube outlet
QDRR = the detention facility maximum release rate in cfs
dO = the nominal dia. of the orifice tube outlet in in.

$$S_{DR} = 1{,}930\ (0.069)^2/(1)^4 + 1/24$$

H= [4.4]9.23 ft

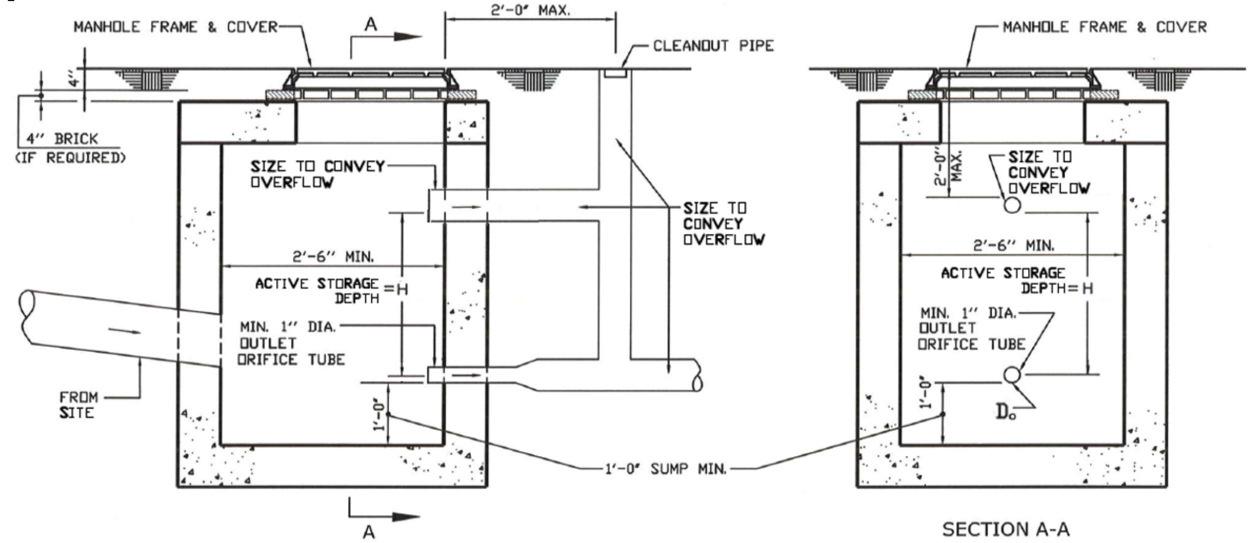
[where:
Q_O = maximum release rate of orifice (cfs) = 0.069 cfs
C_D = coefficient of discharge, 0.52 for re-entrant orifice
A_O = area of orifice (sf) = 0.005 sf
g = acceleration due to gravity, 32.2 (ft/s^2)
H = maximum hydraulic head above the centerline of the orifice (ft)]

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]3.8 ft.

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

Based on the active storage depth of [3.4]3.8 ft and the V_V of 229 cf, set the interior detention tank dimensions to L: [8.5]7.8 ft and W: [8.5]7.8 ft. The resulting detention tank has an active storage volume of [246]231 cf. Note that the exterior dimensions of the detention tank will be larger than the dimensions of the active storage zone ([8.5]7.8'L x [8.5]7.8'W x [3.4]3.8'D) to accommodate wall thickness, bypass structures, and/or other internal features.

[



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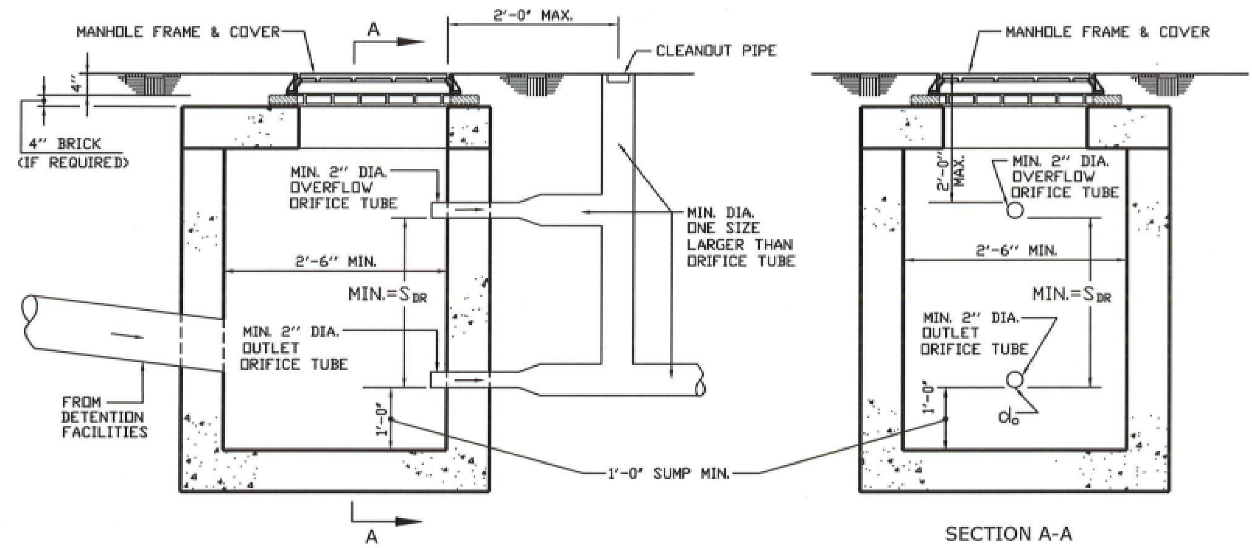


Figure [F]D.5. Outdoor Detention Tank with Re-Entrant Orifice

APPENDIX E

Site Design Example

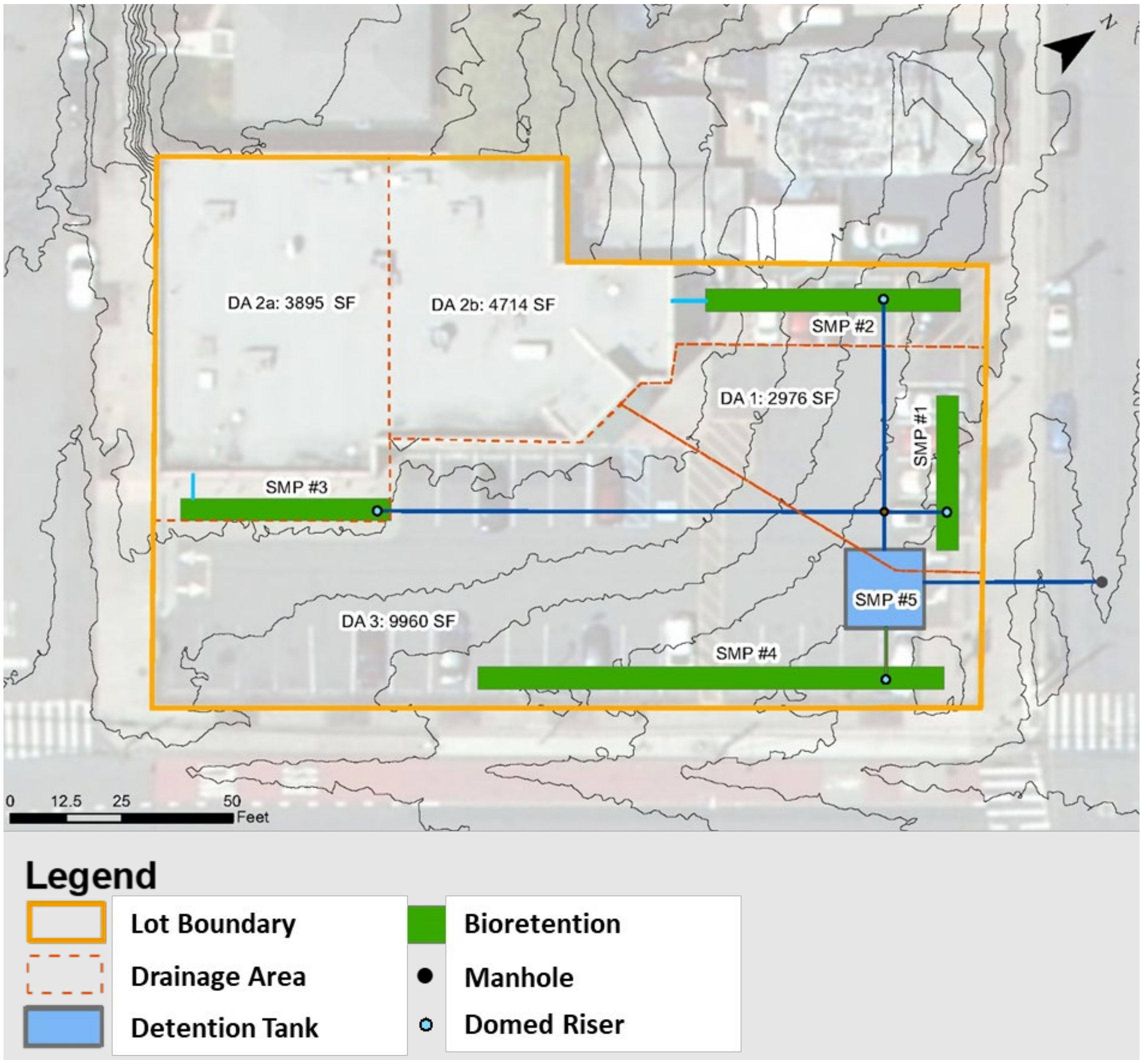
Site Design Example

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

Table [G]E.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.

Compute the maximum storage depth in ft. of a detention facility with a Re-entrant orifice tube outlet, S_{DR}, with a CD of 0.52, by the equation:

$$S_{DR} = 1{,}930\ (Q_{DRR})^2/(d_O)^4 + d_O/24$$

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)]

S_{DR} = the maximum storage depth in ft. for a Re-entrant orifice tube outlet.

Q_{DRR} = the maximum storage depth in ft. for a Re-entrant orifice tube outlet.

d_O = the nominal dia. Of the orifice tube outlet in in.

$$S_{DR} = 1{,}930\ (0.049)^2/(1)^4 + 1/24$$
$$H = 4.68\ ft$$

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]4.68 ft and the $V_{V,Tank}$ of 1,566.65 cf, set the interior detention tank dimensions to L: 18.5[17] ft and W: [17]18.5 ft. The resulting detention tank has an active storage volume of [1,589.5]1,601.7 cf. Note that the exterior dimensions of the detention tank will be larger than the dimensions of the active storage zone (18.5[7]'L x 18.5[7]'W x [5.5]4.68'D) to accommodate wall thickness, bypass structures, and/or other internal features.

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

Table [G]E.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	18.5[7] x 18.5[7] x 4.68[5.5]	1,[589.5]601.7	0	1,589.5
Total	**21,545**	**-**	**-**	**2,558.5**	**2,945.75**

APPENDIX G

Detention in Series Workbook and Examples

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 the accompanying Appendix [I]G workbook Excel file.

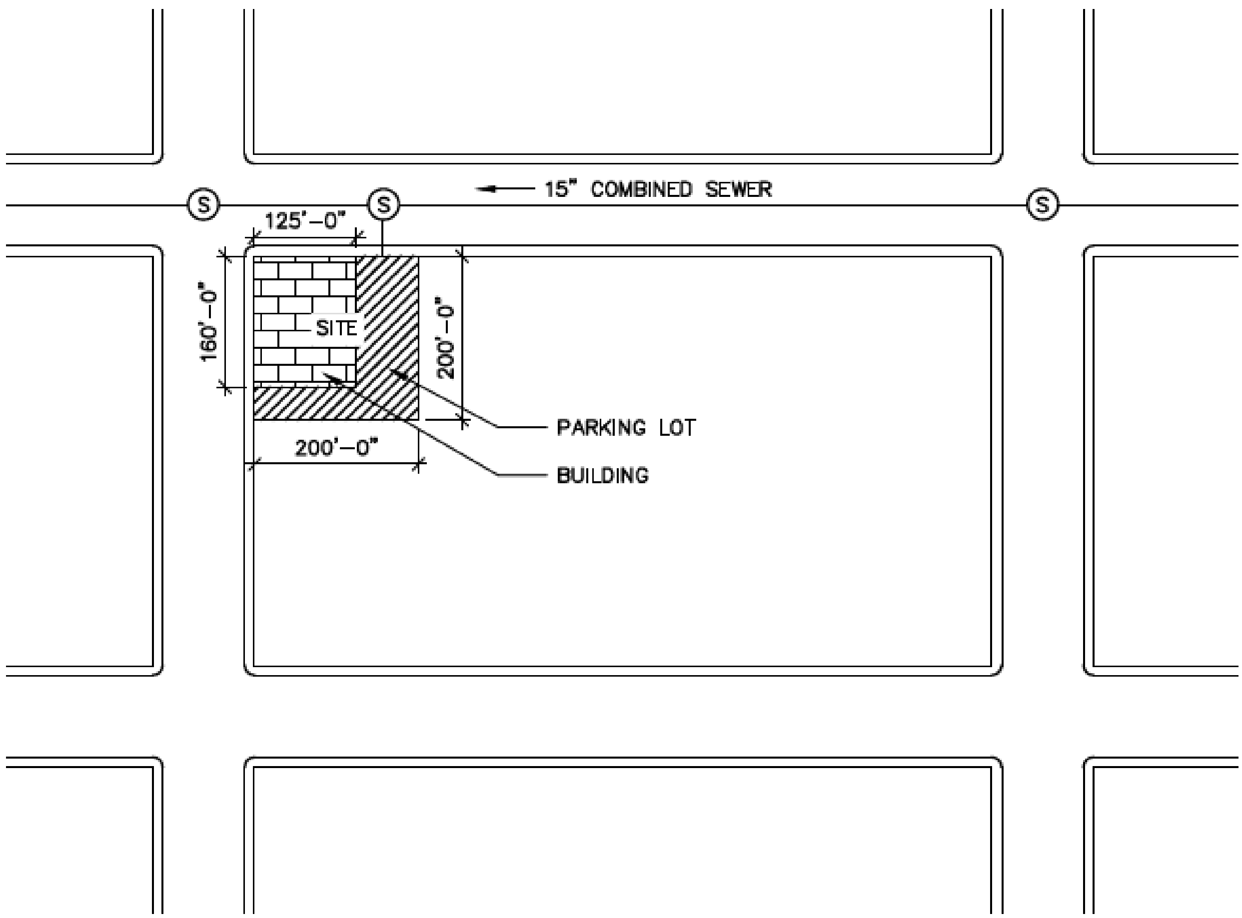


Figure [I]G.1. Schematic of Example 1 (Not to Scale)

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]G.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. Controlled flow roof drains may have a standard flow rate per unit depth, controlled by a parabolic weir, or may have a flow rate through a custom orifice, which has different design requirements.

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]G.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.

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

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.

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

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]G.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] cannot be used for detention, because some roof area tributary to the system will be a bulkhead or parapet, and volume can't be provided in those. Therefore, the available area is [the entire 20,000 sf] set by the designer and must be less than 20,000 sf. For the purpose of this example, it will be assumed that the available area is 19,000 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 configuration, as shown in Figure [I.6]I.5.

[Uni-directionally Sloped Roof:

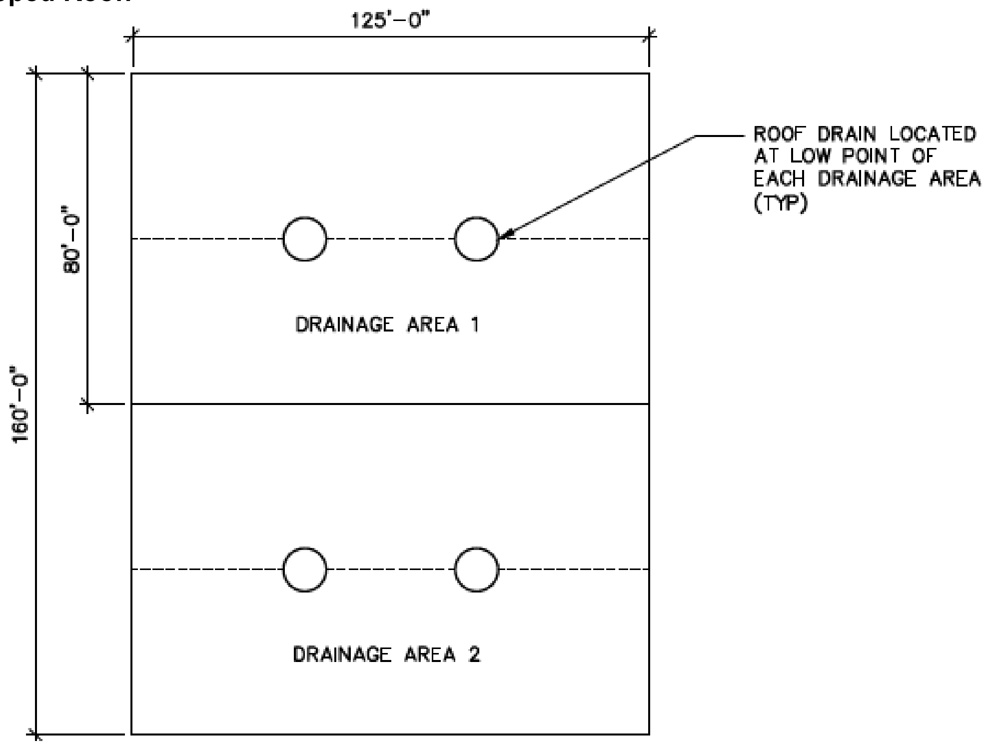


Figure I.5. Plan View of Uni-Directionally Sloped Blue Roof

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

$$V_T = 833\ cf + 833\ cf$$
$$V_T = 1{,}666\ cf \leq V_V = 2{,}414\ cf \qquad \textbf{\textit{NOT MET}}$$

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:]

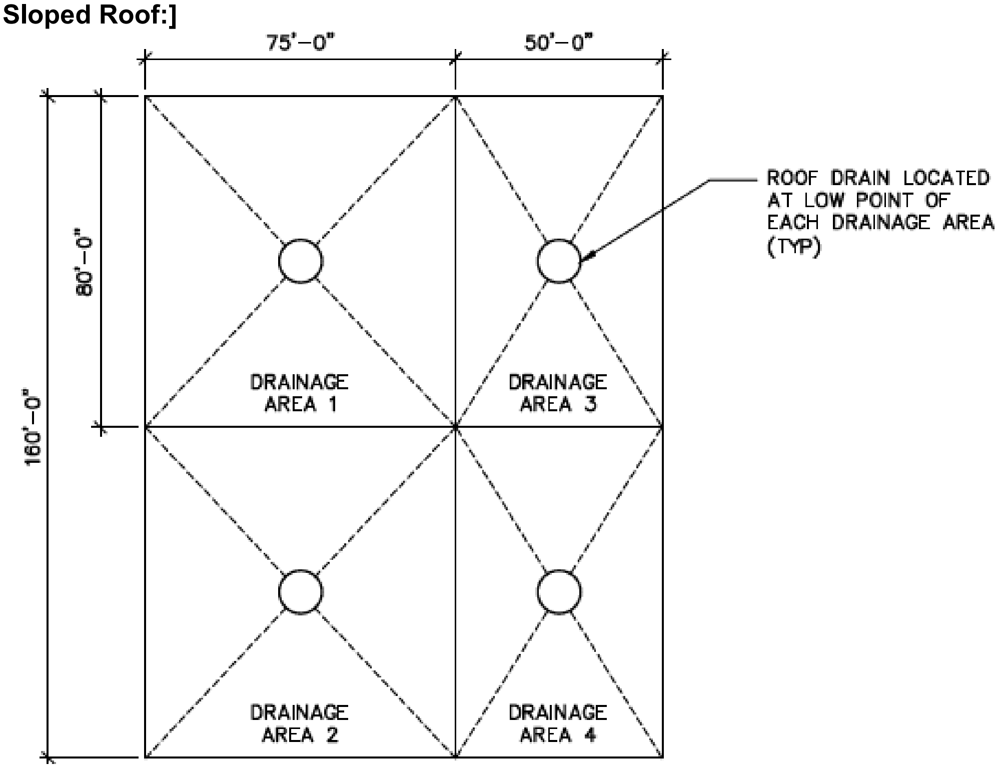


Figure [I.6.]G.5. Plan View of Multi-Directionally Sloped Blue Roof

To calculate the provided volume to meet the operations volume requirement, Vv, BWSO does not analyze the volumes of the tributary areas individually and assumes a best-case scenario for tributary areas. The area available for detention is always less than the tributary area, and it is assumed that the area available for detention here is 18,000 sf.

For the example above, the best case scenario is reflected by determining the[The] weighted average of the length[s] and width[s] of [each]a typical drainage area[are], as follows, assuming a L:W ratio of 1.25:1:

Drainage Area[1]: [75]75'L x [80]60'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, or about 1% 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.]of the short edge of an average drainage area, d_S is 3.6" above the center drain, and the depth to the long edge d_L is 4.5" above a center drain. The roof slope for roof detention should not be less than 0.5%, or 0.0625 in per ft.
Note: the height of any overflow must be between 2" and 4" above the primary drain to meet requirements from the NYC Department of Buildings.
Calculate the available storage volume of [each]an average drainage area, accounting for the best-case scenario and a constant roof slope.
There are three components to the volume provided for an average drain area, depending on the depth of flow.

1) If the depth of flow will be less than or equal to the shortest overflow height, the volume provided is calculated by checking the volume of an inverted pyramid[, using the volume of a pyramid] with base width equal to the width specified above, for example 61.6'.

[Drainage Areas 1 and 2]Typical Drainage Area, Inverted Pyramid:

$$V_A = \frac{1}{3}W^2 * \frac{d_R}{12}$$

where:

V_A = the available storage volume of a typical[each] drainage area (cf)

[L = the length of each drainage area (ft) = 75 ft]

W = the width of [each]a typical drainage area (ft) [= 80 ft]

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

2) If the depth of flow, d_R = the roof drain depth of flow (in) will be between the shortest overflow height and the longest overflow height, the volume provided includes the full inverted pyramid, calculated using 1), and adding the volume in an inverted trapezoidal prism up to the long overflow depth. The volume up to this depth is calculated with the formula:

$$V_A = \frac{1}{3}W^2 \cdot \frac{d_S}{12} + \frac{\left(16.67 \cdot \frac{d_R}{S} + W\right)}{2} W \cdot \frac{(d_R - d_S)}{12}$$

where:

V_A = the available storage volume of a typical drainage area (cf)

W = the width of a typical drainage area (ft)

L = the length of a typical drainage area (ft)

d_R = the roof drain depth of flow (in)

d_S = the depth to the short overflow height (in)

S = the roof slope, in units of percentage (between 0 and 100%)

3) If the depth of flow, d_R = the roof drain depth of flow (in) will be above the longest overflow height, the volume provided includes 1), 2) and adds the volume of a rectangular prism up to the maximum overflow height.

$$V_A = \frac{1}{3}W^2 \cdot \frac{d_S}{12} + \frac{(L + W)}{2} W \cdot \frac{(d_L - d_S)}{12} + (L \cdot W)\frac{\left(d_R - (d_L + d_S)\right)}{12}$$

where:

V_A = the available storage volume of a typical drainage area (cf)

W = the width of a typical drainage area (ft)

L = the length of a typical drainage area (ft)

d_R = the roof drain depth of flow (in)

d_S = the depth to the short overflow height (in)
d_L = the depth to the long overflow height (in)

In the example, using the assumed roof drain of 10 GPM/in/weir, the depth must not exceed 2.0 inches to correctly restrict flow from the facility to 0.18 cfs. Because the depth will not exceed 2.0 inches, the first equation applies:

$$V_A = \frac{1}{3} * 60^2\ ft^2 * \frac{2.0\ in}{12} = 200\ cf$$

This indicates that only 200 cf of volume will be provided in the area of an average inverted pyramid, so with four drains the provided volume would be 800 cf, but the volume required is 2414. This demonstrates that the volume requirements are not being met by the proposed drain system, and as a result, another drain configuration is required.

Since the maximum overflow height is known to be 4” above the primary drain, it is possible to set this as the depth of flow d_R to determine a more appropriate drain. With the depth of flow not to exceed 4”, and the release rate from all drains not to exceed 0.2 cfs at this depth, a target drain flow rate can be determined:

$$Q_{\frac{GPM}{in}, max} = \frac{\left(Q_{drr}\,(cfs) \cdot 448\ \frac{gpm}{cfs}\right)}{\#Drains \cdot Depth\ of\ flow\ (in)}$$

In this example, the maximum flow rate for an individual drain at a depth of 4” should not exceed 5.6 GPM/inch.

With this drain, the provided volume can be calculated using the above relationships, with a flow depth of 4”. This is scenario 2) above, as the flow depth is between the short overflow height and the long overflow height.

$$V_A = \frac{1}{3}(60)^2 \cdot \frac{3.6}{12} + \frac{\left(16.67 \cdot \frac{4}{1} + 60\right)}{2} 60 \cdot \frac{(4 - 3.6)}{12} = 486.7\ cf$$

For a system with four drains, this has a provided volume of ~1,946 cf, which is less than the required volume of 2,414.

[Drainage Areas 3 and 4:

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

The total available storage volume is:

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]

Since the required detention volume is greater than the available storage volume, the user may select a different controlled-flow roof drain or design depth of flow and re-run Steps 2-4.
In this case, there is no circumstance where a drain can restrict flow sufficiently below the overflow to meet the volume requirements, so the designer has must choose a different drain, roof slope, or release rate from the roof detention system. With a roof slope of 0.5%, the system has enough volume, but the roof slope is not always easily adjusted.
In the case when the roof slope may not be adjusted: If the flow depth is chosen to be [3]4 inches as the new design depth of flow, a drain that discharges at a flow rate of 5 GPM/in is selected. The new ponding depth results in a maximum release rate of 0.[27]43 cfs, a required detention volume of [2,242]1,938 cf, and a total available storage volume of [1,666]1,947 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]1,938 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	

]

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	0.43	1938	

Figure [I.7.]G.6. 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.

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

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

$$C_{WE} = \frac{311 * 0.43\, cfs * (56.6\, min + 15)}{20{,}000\, sf}$$

$$C_{WE} = 0.380$$

[

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

]

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	0.43	1938	0.38

Figure [I.8.]G.7. 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

]

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	0.43	1938	0.38
2	20000	0.85	None			0.85

Figure [I.9.]G.8. 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

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

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.]G.9. 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.

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

where:

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

C_1 = the effective weighted runoff coefficient for the area classified as roof = 0.3[0]8

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

$$C_W = \frac{(0.38 * 20{,}000 \ sf) + (0.85 * 20{,}000 \ sf)}{40{,}000 \ sf}$$

$$C_W = 0.61$$

[

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

]

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		0.61

Figure [I.11.]G.10. 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]61

$$V_V = \frac{1.85\ in}{12} * 40{,}000\ sf * 0.61$$

$$V_V = 3{,}791\ cf$$

[

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

]

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	3791	0.61

Figure [I.12.]G.11. 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.

Compute the maximum storage depth in ft. of a detention facility with a Re-entrant orifice tube outlet, S_{DR}, with a C_D of 0.52, by the equation:

$$S_{DR} = 1{,}930\ (Q_{DRR})^2/(d_o)^4 + d_o/24$$

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)]
S_{DR}[S] = the maximum storage depth in ft. For a Re-entrant orifice tube outlet
Q_{DRR} = the detention facility maximum release rate in cfs.
[Q_{DRR}]d_O= the [detention facility maximum release rate in cfs.]nominal dia. of the orifice tube outlet in in.

$$S_{DR} = 1{,}930\ (0.092)^2/(1.25)^4 + 1.25/24$$
$$H = 6.74\ ft$$

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]91 ft.

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

Based on the active storage depth of 4.91[4] ft and the V_V of 3,[548]791 cf, set the interior detention tank dimensions to L: 28.[5]0 ft and W: 2[8.5]0 ft. The resulting detention tank has an active storage volume of 3,[574]849.44 cf. Note that the exterior dimensions of the detention tank will be larger than the dimensions of the active storage zone (28.[5]0'L x 28.[5]0'W x 4.[4]91'D) to accommodate wall thickness, bypass structures, and/or other internal features.