Appendix D
STORMWATER MANAGEMENT PRACTICE SIZING EXAMPLES

NYC Environmental Protection

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 WQv.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

$$V_{SMP} = 59.4 \; cf < WQ_V = 130.63 \; cf \qquad \textbf{\textit{NOT MET}}$$

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) = $(\pi) * (0.5)^2$ = 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

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

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

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

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

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

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

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

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) = $(\pi) * (0.5)^2$ = 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

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

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

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

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

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

Surface ponding drawdown time:

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

where:

dt_P = drawdown time of surface ponding (hr)

V_P = volume of surface ponding (cf) = 1,969.29 cf

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

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

D_m = depth of media below surface ponding area (ft) = 3.5 ft

A_{P1} = area at the base of surface ponding zone (sf) = 1,400 sf

A_{P2} = area at the top of surface ponding zone (sf) = 2,600 sf

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

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

SEWER OPERATIONS
VOLUME SIZING EXAMPLES

Detention

Detention Tank - CSS with SCP

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

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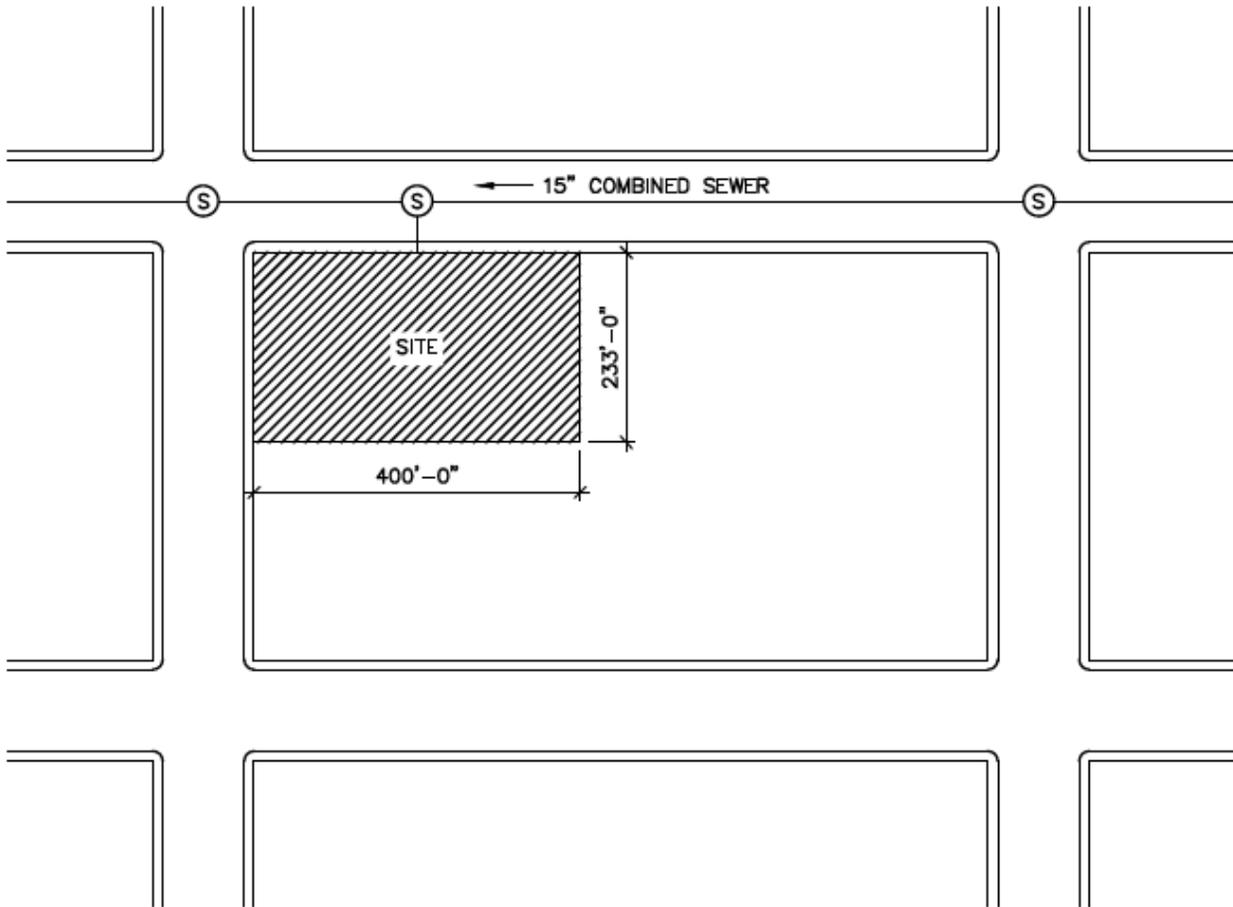Grass = 16,200 sf @ 0.20 runoff coefficient

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

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

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

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

R$_D$	Description
1.85	CSS areas with SCP
1.50	CSS areas with HCP
1.50	MS4 areas wiith SCP
1.10	MS4 areas with HCP

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

Step 2: Calculate the runoff coefficient (C$_W$) using the weighted area approach.

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

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) = 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, $q = 0.1\ \frac{cfs}{acre}$.

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

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

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

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

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/s²)

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

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

$$A_o = 0.026 \; sf$$

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

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

where:

A_O = area of orifice (sf) = 0.026 sf

D_O = diameter of orifice (in)

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

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

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

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

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

where:

A_O = area of orifice (sf)

D_O = diameter of orifice (in) = 2 in

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

$$A_O = 0.022 \; sf$$

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

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

where:

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

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

A_O = area of orifice (sf) = 0.022 sf

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

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

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

$$H = 5.4\ ft$$

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

In this case, the depth is feasible.

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

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

Detention Tank - CSS with HCP

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

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

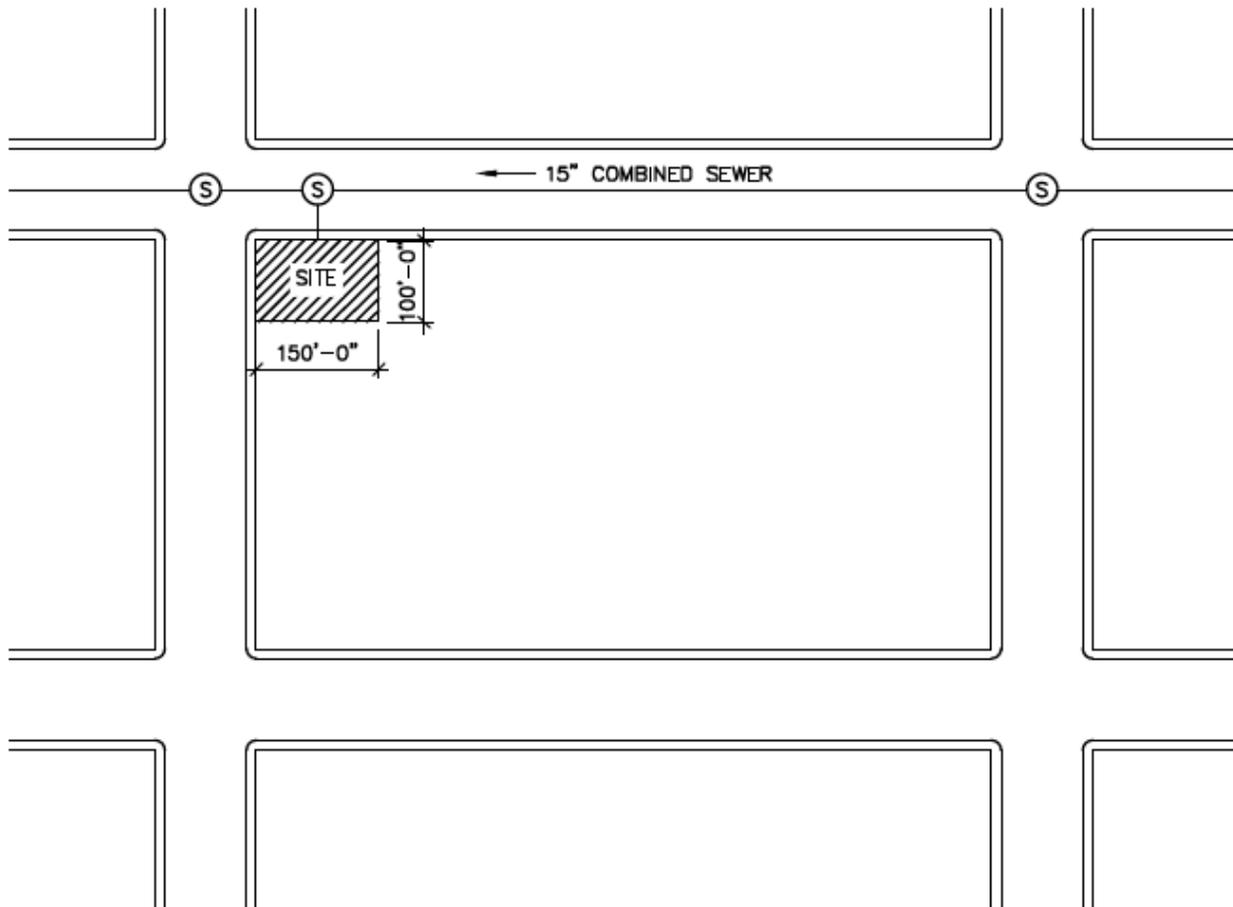**Figure F.2. Schematic of Site (Not to Scale)**

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

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

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

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

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

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

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

where:

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

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

A_1 = the area classified as roof (sf) = 2,000 sf

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

A_2 = the area classified as paved (sf) = 7,000 sf

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

A_3 = the area classified as grass (sf) = 6,000 sf

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

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

$$C_W = 0.603$$

Step 3: Calculate V_V.

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

where:

V_V = sewer operations volume (cf)

R_D = rainfall depth (in) = 1.50 in

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

C_W = weighted runoff coefficient relating peak rate of rainfall and runoff = 0.603

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

$$V_V = 1{,}131\ cf$$

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

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

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

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

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

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

where:

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

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

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

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

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

The maximum release rate is 0.046 cfs.

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

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

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

where:

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

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

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

$$A_o = 0.006 \; sf$$

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

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

where:

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

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

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

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

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

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

where:

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

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

$A_O = 0.005 \; sf$

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

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

where:

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

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

A_O = area of orifice (sf) = 0.005 sf

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

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

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

$H = 4.9 \; ft$

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

In this case, the depth is feasible.

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

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

Detention Tank - MS4 with SCP

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

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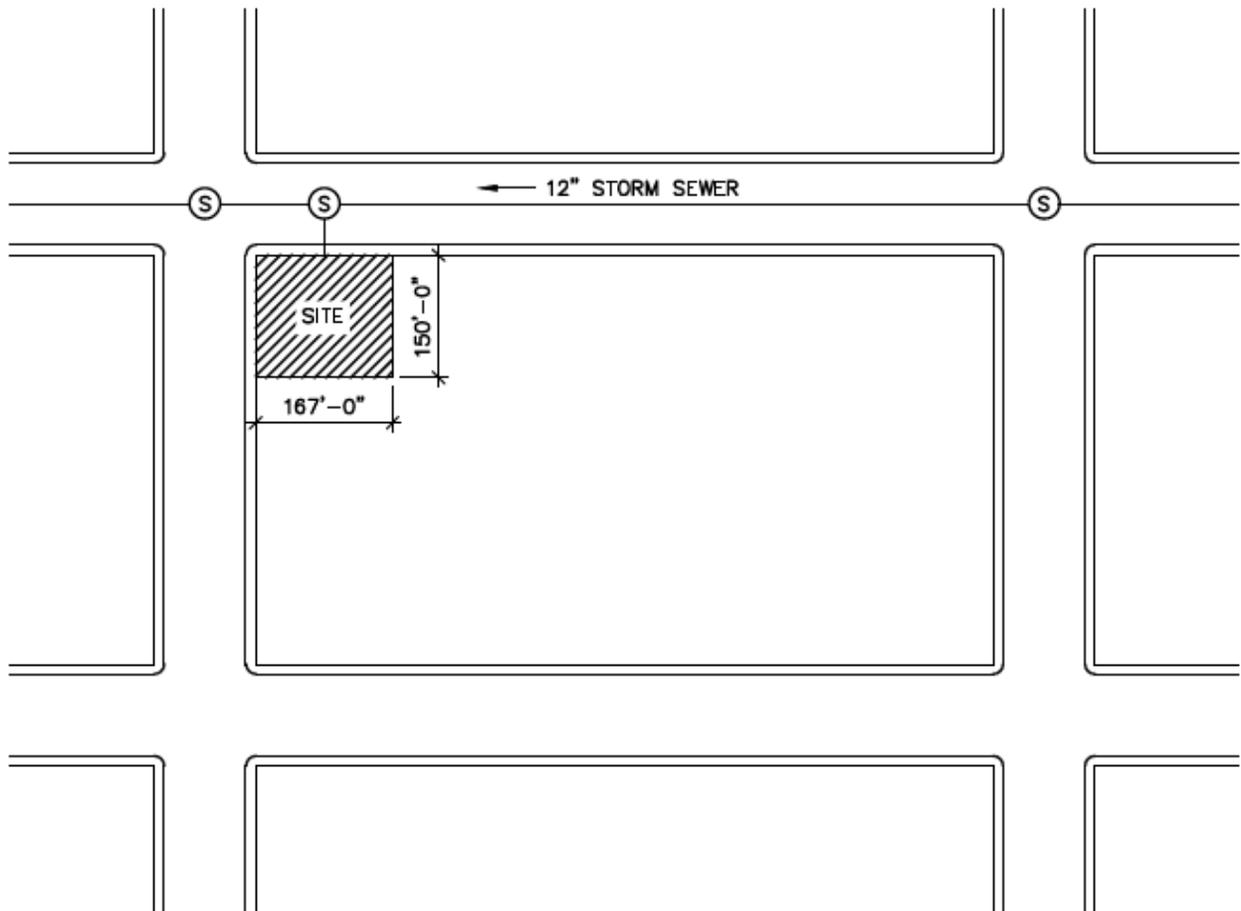Grass = 2,950 sf @ 0.20 runoff coefficient

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

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

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

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

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

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

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

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

where:

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

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

A_1 = the area classified as roof (sf) = 16,000 sf

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

A_2 = the area classified as paved (sf) = 6,100 sf

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

A_3 = the area classified as grass (sf) = 2,950 sf

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

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

$$C_W = 0.837$$

Step 3: Calculate V_V.

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

where:

V_V = sewer operations volume (cf)

R_D = rainfall depth (in) = 1.50 in

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

C_W = weighted runoff coefficient relating peak rate of rainfall and runoff = 0.837

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

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

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

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

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

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

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

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

where:

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

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

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

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

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

The maximum release rate is 0.575 cfs.

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

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

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

where:

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

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

A_O = area of orifice (sf)

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

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

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

$$A_o = 0.069 \; sf$$

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

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

where:

A_O = area of orifice (sf) = 0.069 sf

D_O = diameter of orifice (in)

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

$$D_O = 3.56 \; in \; > 1 \; in \quad OK$$

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

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

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

where:

A_O = area of orifice (sf)

D_O = diameter of orifice (in) = 3.50 inches

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

$$A_O = 0.067 \; sf$$

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

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

where:

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

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

A_O = area of orifice (sf) = 0.067 sf

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

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

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

$$H = 4.2 \; ft$$

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

In this case, the depth is feasible.

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

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

Detention Tank - MS4 with HCP

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

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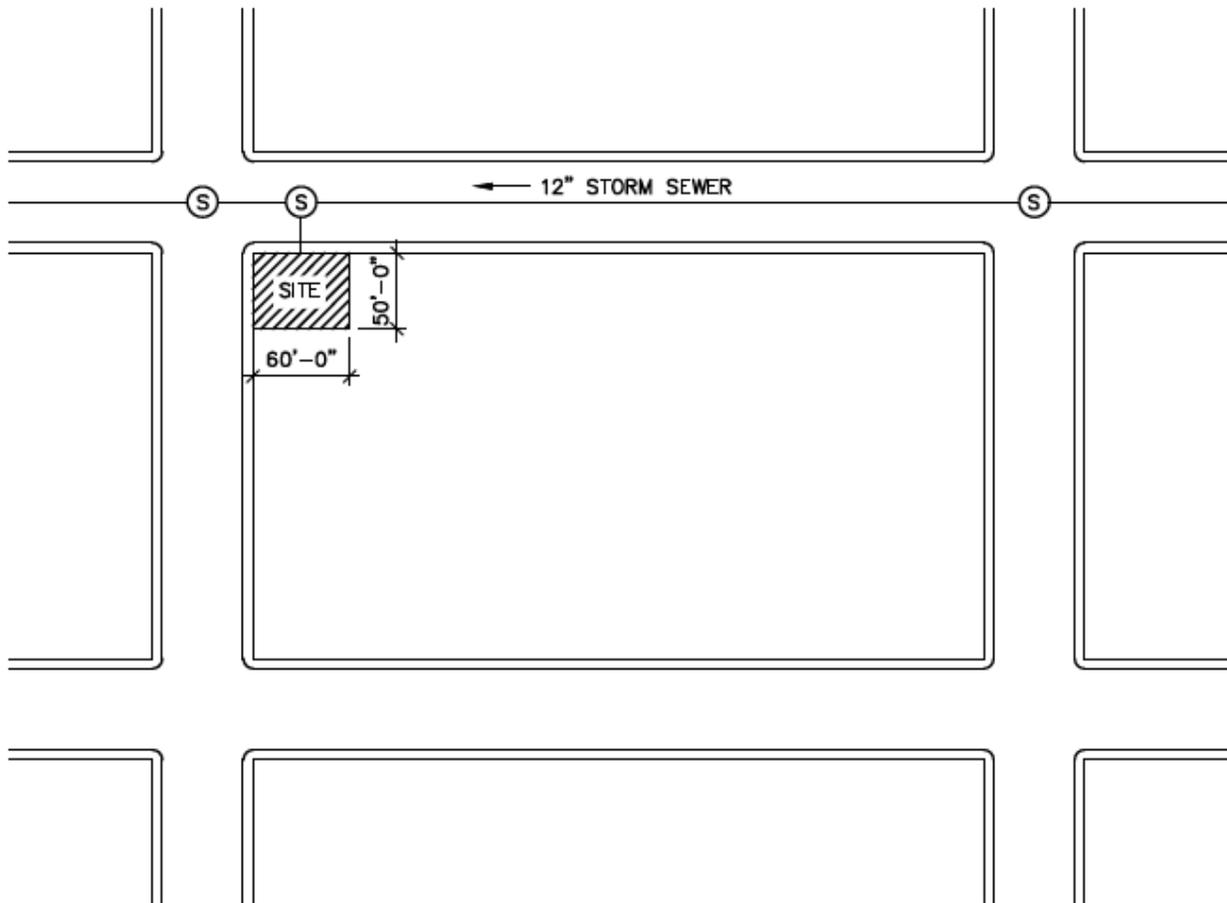Grass = 400 sf @ 0.20 runoff coefficient

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

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

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

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

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

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

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

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

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/s²)

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)

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

$$D_O = 1.21 \; in \; > 1 \; in \quad OK$$

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

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

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

where:

A_O = area of orifice (sf)

D_O = diameter of orifice (in) = 1 inch

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

$$A_O = 0.005 \; sf$$

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

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

where:

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

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

A_O = area of orifice (sf) = 0.005 sf

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

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

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

$$H = 10.9 \; ft$$

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

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

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

Based on the active storage depth of 3.4 ft and the V_V of 229 cf, set the interior detention tank dimensions to L: 8.5 ft and W: 8.5 ft. The resulting detention tank has an active storage volume of 246 cf. Note that the exterior dimensions of the detention tank will be larger than the dimensions of the active storage zone (8.5'L x 8.5'W x 3.4'D) to accommodate wall thickness, bypass structures, and/or other internal features.
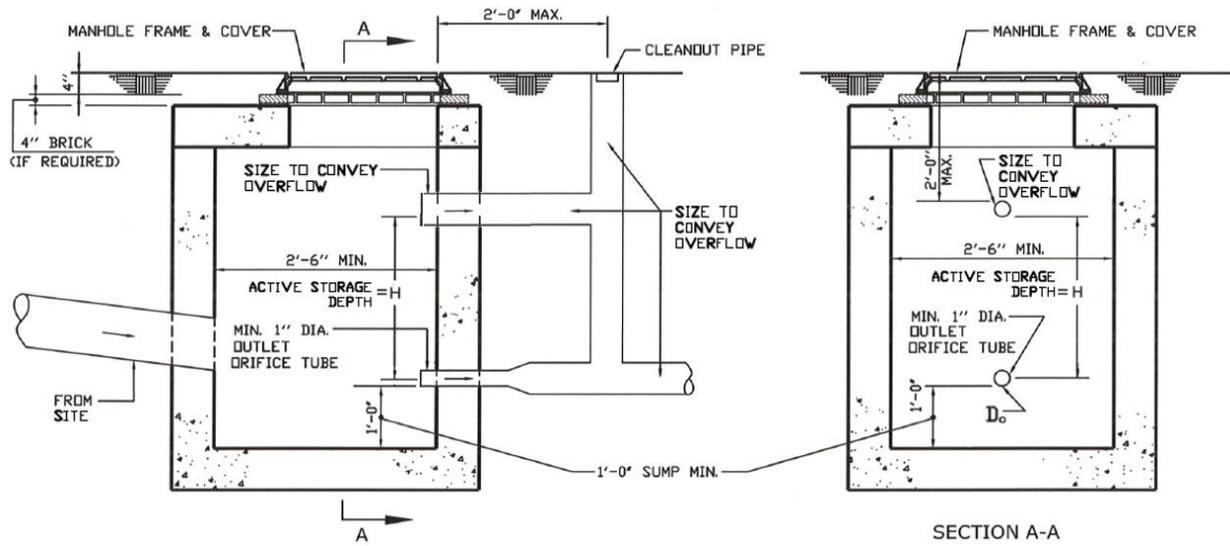
Figure F.5. Detention Tank with Re-Entrant Orifice