

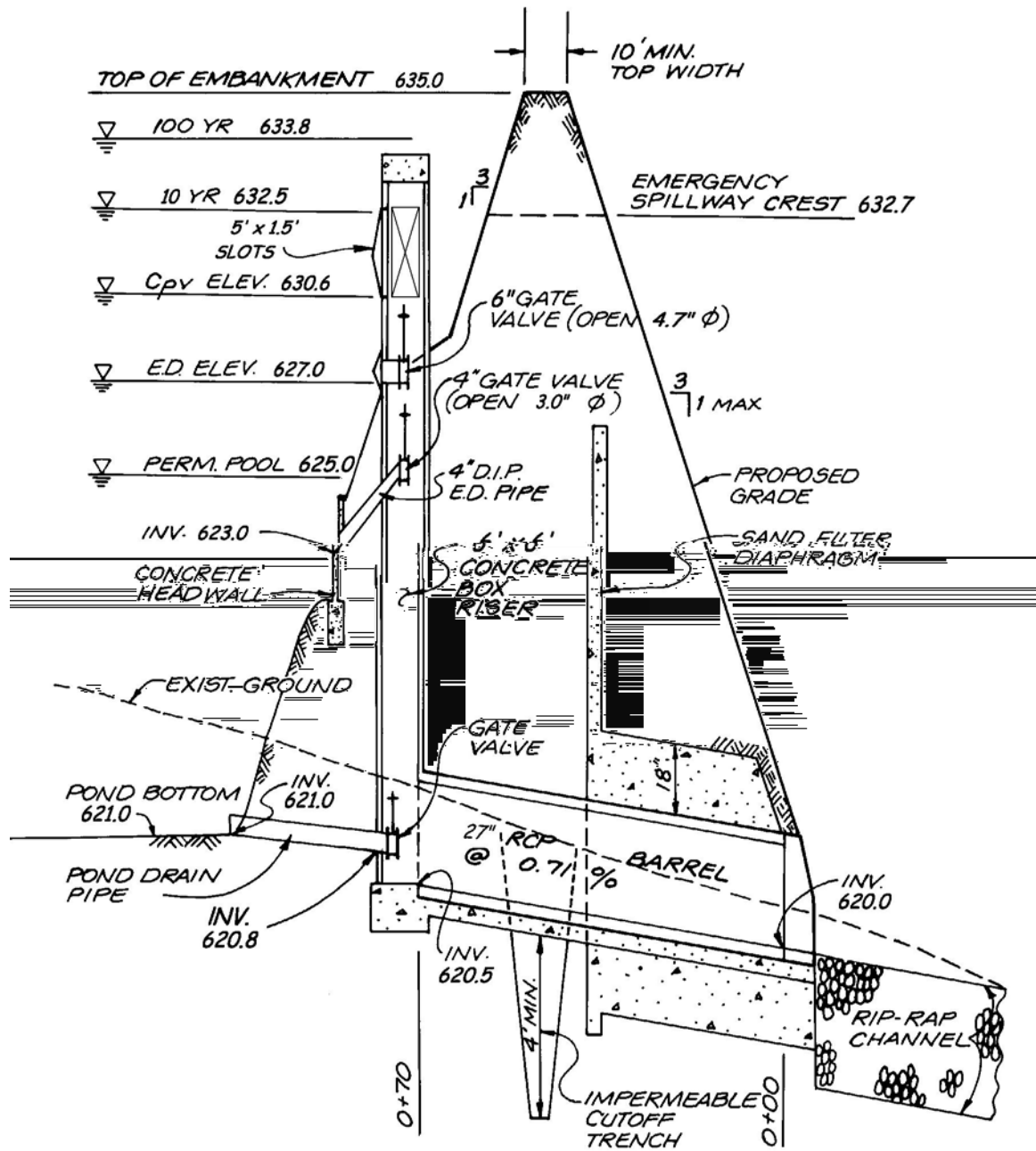
Figure 8.12 TR-20 Model Input and Output for Ultimate Buildout Conditions

TR20 XEQ 1/22/** New York Manual Wet ED Example 1/01 EWB JOB 1 SUMMARY
 REV 09/01/83 Ultimate Buildout Conditions for 100-yr PAGE 4

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
 (A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
 A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION				RUNOFF AMOUNT (IN)	PEAK DISCHARGE			
						BEGIN (HR)	AMOUNT (IN)	DURATION (HR)	ELEVATION (FT)		TIME (HR)	RATE (CFS)	RATE (CSM)	
ALTERNATE	1	STORM	99											
STRUCTURE	1	RUNOFF	.10	2	2	.10	.0	5.50	24.00	3.53	---	12.10	230.71	2261.9
STRUCTURE	1	RESVOR	.10	2	2	.10	.0	5.50	24.00	3.19	634.00	12.22	191.83	1880.7

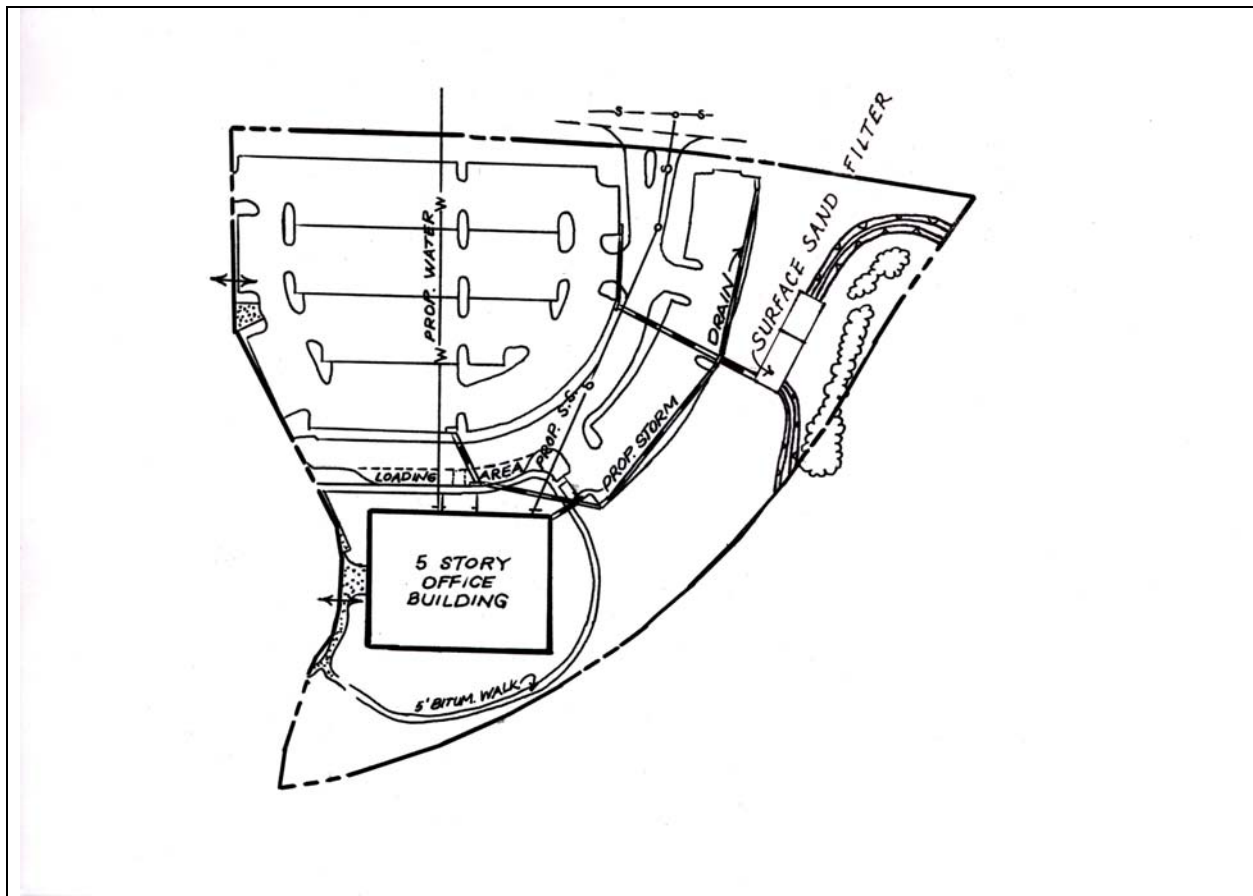
Figure 8.13 Profile of Principle Spillway



Section 8.3 Sand Filter Design Example

This design example focuses on the design of a sand filter for a 4.5-acre catchment of Lake Center, a hypothetical commercial site located in Albany, NY. A five-story office building and associated parking are proposed within the catchment. The layout is shown in Figure 8.14. The catchment has 3.05 acres of impervious cover, resulting in 68% impervious cover. The pre-developed site is a mixture of forest and meadow. On-site soils are predominantly HSG “B” soils.

Figure 8.14 Lake Center Site Plan



<p><u>Base Data</u> Location: Albany, NY Site Area = Total Drainage Area (A) = 4.50 ac Impervious Area = 3.05 ac; or $I = 3.05/4.50 = 68\%$ Soils Type “B”</p>	<p style="text-align: center;"><i>Hydrologic Data</i></p> <table border="1" style="margin-left: auto; margin-right: auto;"> <thead> <tr> <th></th> <th>Pre</th> <th>Post</th> </tr> </thead> <tbody> <tr> <td>CN</td> <td>58</td> <td>83</td> </tr> <tr> <td>t_c (hr)</td> <td>.44</td> <td>.10</td> </tr> </tbody> </table>		Pre	Post	CN	58	83	t_c (hr)	.44	.10
	Pre	Post								
CN	58	83								
t_c (hr)	.44	.10								

This step-by-step example will focus on meeting the water quality requirements. Channel protection control, overbank flood control, and extreme flood control are not addressed in this example. Therefore, a detailed hydrologic analysis is not presented. For an example of detailed sizing calculations, consult section 8.1. In general, the primary function of sand filters is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. For this example, the post-development 10-yr peak discharge is provided to appropriately size the necessary by-pass flow splitter. Where quantity control is required, bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults).

Step 1. Compute design volumes using the Unified Stormwater Sizing Criteria.

Water Quality Volume, WQ_v

Select the Design Storm

Consulting Figure 4.1 of this document, use 1.0" as the 90% rainfall event for Albany.

Compute Runoff Coefficient, R_v

$$R_v = 0.05 + (68)(0.009) = 0.66$$

Compute WQ_v

$$\begin{aligned} WQ_v &= (1.0") (R_v) (A) / 12 \\ &= (1.0") (0.66) (4.5 \text{ ac}) (43,560 \text{ ft}^2/\text{ac}) (1 \text{ ft}/12 \text{ in}) \\ &= \underline{10,781 \text{ ft}^3} = \underline{0.25 \text{ ac-ft}} \end{aligned}$$

Develop Site Hydrologic Input Parameters and Perform Preliminary Hydrologic Calculations (see Table 8.3)

Note: For this design example, the 10-year peak discharge is given and will be used to size the bypass flow splitter. Any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations.

Table 8.3 Site Hydrology					
Condition	CN	Q₁	Q₂	Q₁₀	Q₁₀₀
		<i>cfs</i>	<i>cfs</i>	<i>cfs</i>	<i>cfs</i>
Pre-developed	58	0.2	0.4	3	9
Post-Developed	83	7	10	19	36

Step 2. Determine if the development site and conditions are appropriate for the use of a surface sand filter.

Site Specific Data:

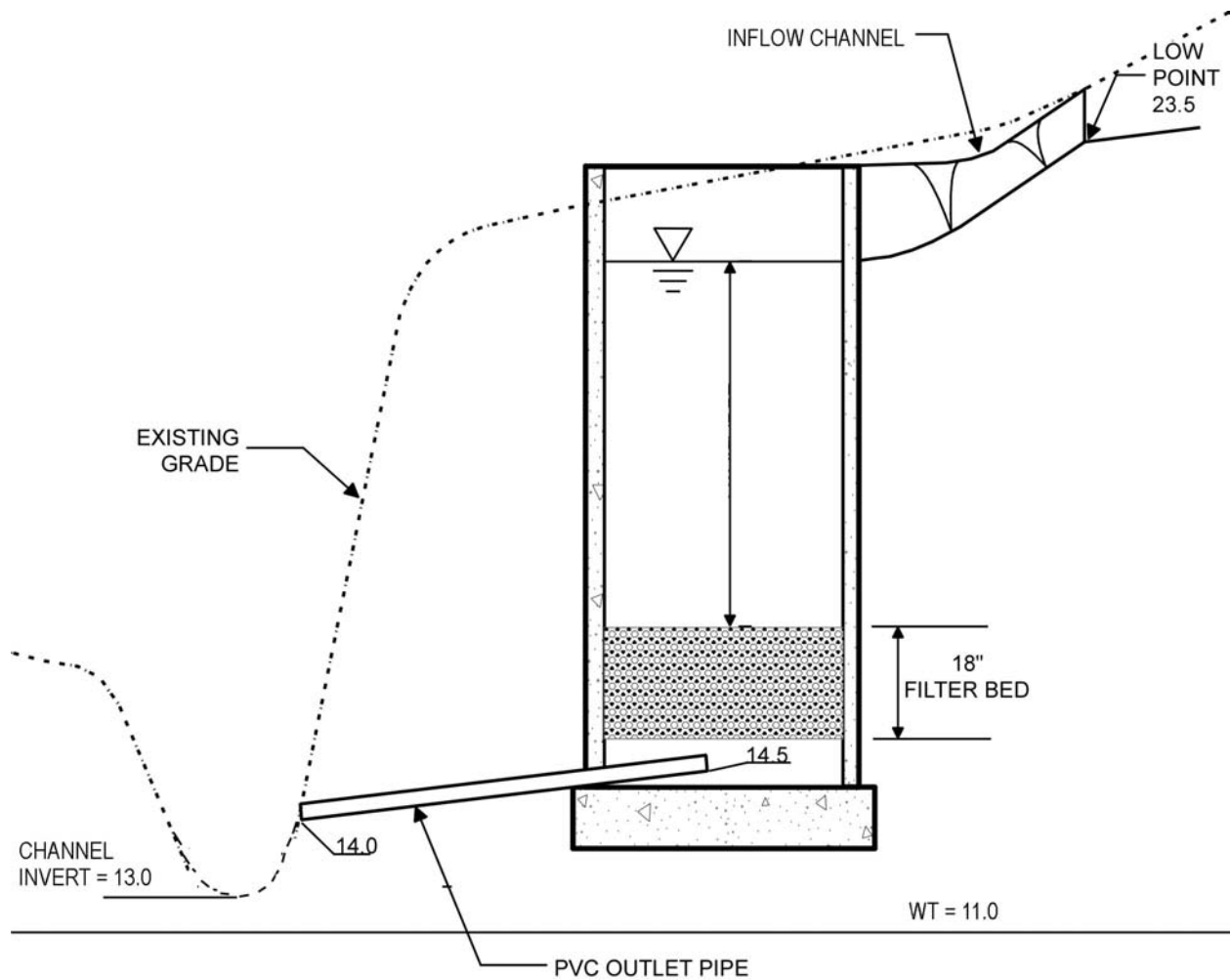
Existing ground elevation at practice location is 222.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 211.0 feet. Adjacent drainage channel invert is at 213.0 feet.

Step 3. Compute available head, & peak discharge (Q_{wq}).

- Determine available head (See Figure 8.15)

The low point at the parking lot is 223.5. Subtract 2' to pass the Q₁₀ discharge (221.5) and a half foot for the inflow channel to the facility (221.0). The low point at the channel invert is 213.0. Set the outfall underdrain pipe 1.0' above the drainage channel invert and add 0.5' to this value for the drain slope (214.5). Add to this value 8" for the gravel blanket over the underdrains, and 18" for the sand bed (216.67). The total available head is 221.0 - 216.67 or 4.33 feet. Therefore, the available average depth (h_f) = $4.33' / 2 = 2.17'$.

Figure 8.15 Available Head Diagram



- Compute Peak Water Quality Discharge:

The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as sand filters and grass channels. The Small Storm Hydrology Method presented in Appendix B was followed to calculate a modified curve number and subsequent peak discharge associated with the 1.0-inch rainfall. Calculation steps are provided below.

Compute modified CN for 1.0" rainfall

$$P = 1.0''$$

$$Q_a = WQ_v \div \text{area} = (10,781 \text{ ft}^3 \div 4.5 \text{ ac} \div 43,560 \text{ ft}^2/\text{ac} \times 12 \text{ in}/\text{ft}) = 0.66''$$

$$CN = 1000/[10+5P+10Q_a-10(Q_a^2+1.25*Q_a*P)^{1/2}]$$

$$= 1000/[10+5*1.0+10*0.66-10(0.66^2+1.25*0.66*1.0)^{1/2}]$$

$$= 96.4$$

Use CN = 96

For CN = 96 and the $t_c = 0.1$ hours, compute the Q_{wq} for a 1.0" storm. With the CN = 96, a 1.0" storm will produce 0.6" of runoff. From TR-55 Chapter 2, Hydrology, $I_a = 0.083$, therefore:

$$I_a/P = 0.083/1.0 = 0.083.$$

From TR-55 Chapter 4 $q_u = 1000$ csm/in, and

$$Q_{wq} = (1000 \text{ csm/in}) (4.5 \text{ ac}/640\text{ac/sq mi.}) (0.66") = \underline{4.6 \text{ cfs.}}$$

Step 4. Size the flow diversion structure.

Assume that flows are diverted to a diversion structure (Figure 8.16). First, size a low-flow orifice to pass the water quality storm ($Q_p = 4.6$ cfs).

$$Q = CA(2gh)^{1/2}; 4.6 \text{ cfs} = (0.6) (A) [(2) (32.2 \text{ ft/s}^2) (1.5')^{1/2}]$$

$$A = 0.77 \text{ sq ft} = \pi d^2/4; d = 0.99' \text{ or } \underline{12"}$$

Size the 10-year overflow as follows:

The 10-year wsel is initially set at 223.0. Use a concrete weir to pass the 10-year flow (19.0 cfs), minus the flow carried by the low flow orifice, into a grassed overflow channel using the Weir equation. Assume 2' of head to pass this event. Overflow channel should be designed to provide sufficient energy dissipation (e.g., riprap, plunge pool, etc.) so that there will be non-erosive velocities.

Determine the flow from the low-flow orifice (Q_{lf}). Assume 3.5' of head (1.5' plus 2' for the 10-year head):

$$Q_{lf} = (0.6) (A) [(2) (32.2 \text{ ft/s}^2) (3.5')^{1/2}]$$

$$A = \pi (1')^2/4$$

$$= 0.78 \text{ sf}$$

So,

$$Q_{fr} = (0.6) (0.78) [(2) (32.2 \text{ ft/s}^2) (3.5')^{1/2}]$$

$$= 7.0 \text{ cfs}$$

Thus, determine the flow passed to the through the channel as:

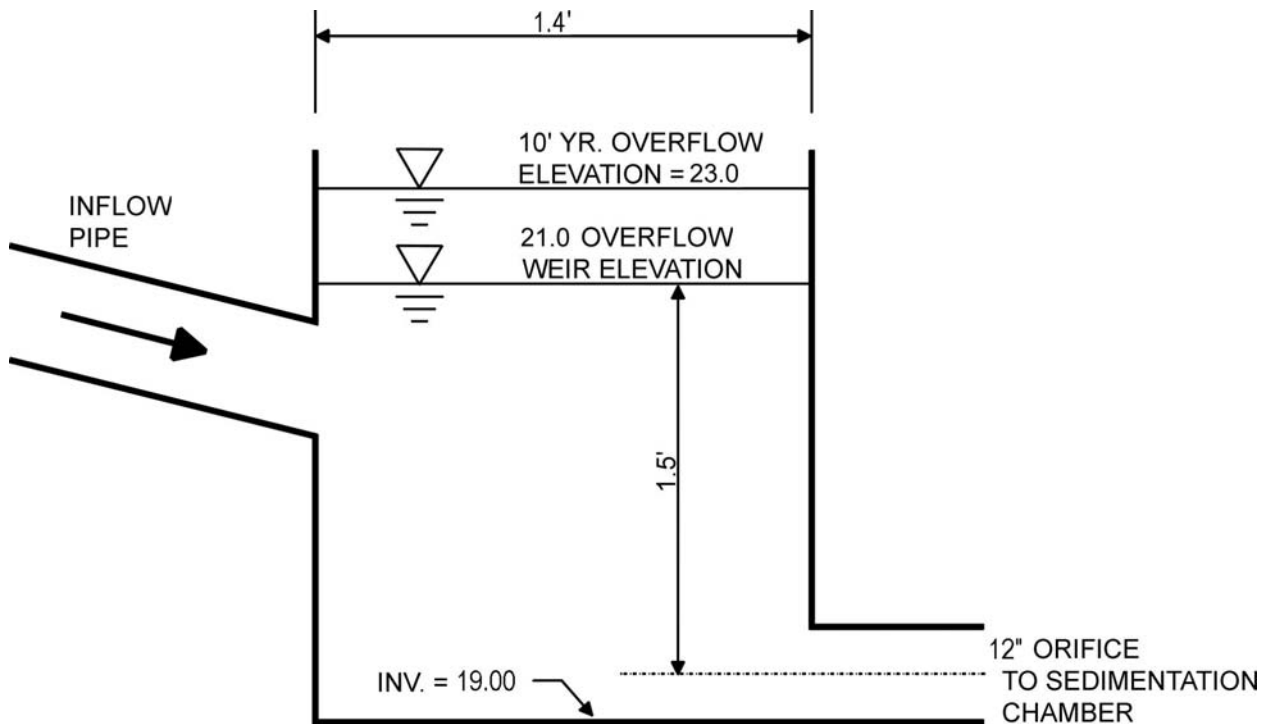
$$Q = CLH^{3/2}$$

$$(19-7) = 3.1 (L) (2')^{1.5}$$

$L = 1.4'$ which sets the minimum length of the flow diversion overflow weir.

Weir wall elev. = 21.0. Set low flow invert at $21.0 - [1.5' + (0.5 * 12" * 1 \text{ ft} / 12")]$ = 19.00.

Figure 8.16 Flow Diversion Structure



Step 5. Size filtration bed chamber (see Figure 8.17).

From Darcy's Law: $A_f = WQ_v (d_f) / [k (h_f + d_f) (t_f)]$

where $d_f = 18"$ or $1.5'$ (Filter thickness)

$k = 3.5 \text{ ft/day}$ (Flow-through rate)

$h_f = 2.17'$ (Average head on filter)

$t_f = 40 \text{ hours}$ (Drain time)

$$A_f = (10,781 \text{ cubic feet}) (1.5') / [3.5 (2.17' + 1.5') (40\text{hr}/24\text{hr}/\text{day})]$$

$$A_f = \underline{755 \text{ sq ft}}; \text{ filter is } \underline{20' \text{ by } 40'} (= 800 \text{ sq ft})$$

Step 6. Size sedimentation chamber.

Size the sedimentation chamber as wet storage with a 2.5' depth. Determine the pretreatment volume as:

$$\begin{aligned} P_v &= (0.25) (10,781 \text{ cf}) \\ &= 2,695 \text{ cf} \end{aligned}$$

Therefore,

$$\begin{aligned} A_s &= (2,695 \text{ cf}) / (2.5') \\ &= 1,078 \text{ sf} \quad (\text{Use } 20' \times 55' \text{ or } 1,100 \text{ sf}) \end{aligned}$$

Step 7. Compute V_{\min} .

$$V_{\min} = \frac{3}{4}(WQ_v) \text{ or } 0.75 (10,781 \text{ cubic feet}) = \underline{8,086 \text{ cubic feet}}$$

Step 8. Compute volume within practice.

Volume within filter bed (V_f): $V_f = A_f (d_f) (n)$; $n = 0.4$ for sand

$$V_f = (800 \text{ sq ft}) (1.5') (0.4) = \underline{480 \text{ cf}}$$

temporary storage above filter bed ($V_{f\text{-temp}}$): $V_{f\text{-temp}} = 2h_f A_f$

$$V_{f\text{-temp}} = 2 (2.17') (800 \text{ sq ft}) = \underline{3,472 \text{ cf}}$$

Compute storage in the sedimentation chamber (V_s):

$$V_s = (2.5')(1,100 \text{ sf}) + 4.33'(1,100 \text{ sf}) = 7,513 \text{ cf}$$

$$V_f + V_{f\text{-temp}} + V_s = 480 \text{ cf} + 3,472 \text{ cf} + 7,513 \text{ cf} = 11,465 \text{ cf}$$

$$11,465 > 8,086 \quad \text{OK.}$$

Pass flow through to the distribution chamber using a 12" orifice with an inverted elbow (see Figure 8.17).

Step 9. Compute sedimentation chamber and filter bed overflow weir sizes.

Assume overflow that needs to be handled is equivalent to the 12" orifice discharge under a head of 3.5 ft (i.e., the head in the diversion chamber associated with the 10-year peak discharge).

$$Q = CA(2gh)^{1/2}$$

$$Q = 0.6(0.79 \text{ ft}^2)[(2)(32.2 \text{ ft/s}^2)(3.5 \text{ ft})]^{1/2}$$

$$Q = 7.1 \text{ cfs}$$

Size the overflow weir from the sediment chamber and the filtration chamber to pass 7.1 cfs (this assumes no attenuation within the practice).

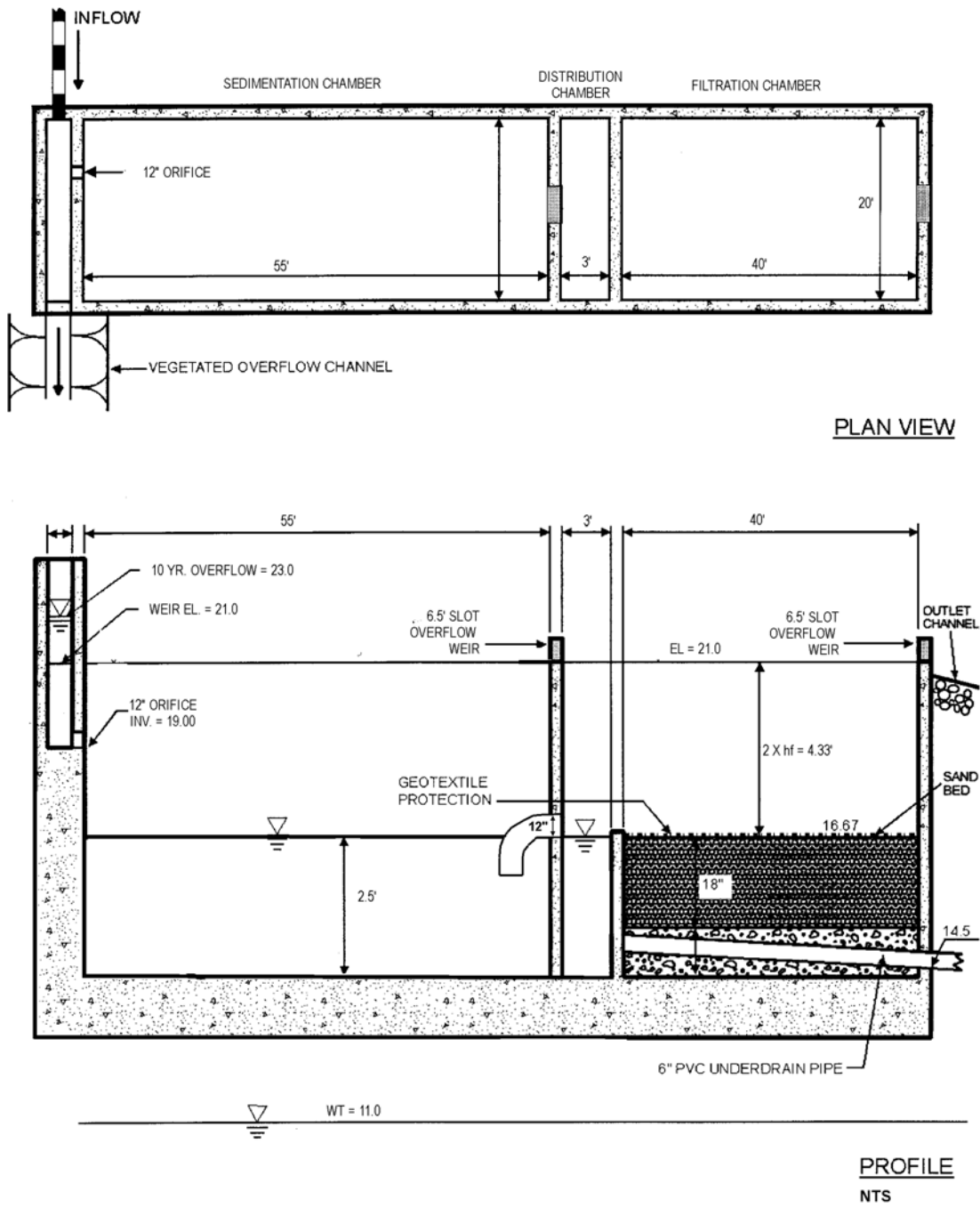
Weir equation: $Q = CLh^{3/2}$, assume a maximum allowable head of 0.5'

$$7.1 = 3.1 * L * (0.5 \text{ ft})^{3/2}$$

$$\underline{L = 6.5 \text{ ft.}}$$

Adequate outlet protection and energy dissipation (e.g., riprap, plunge pool, etc.) should be provided for the downstream overflow channel.

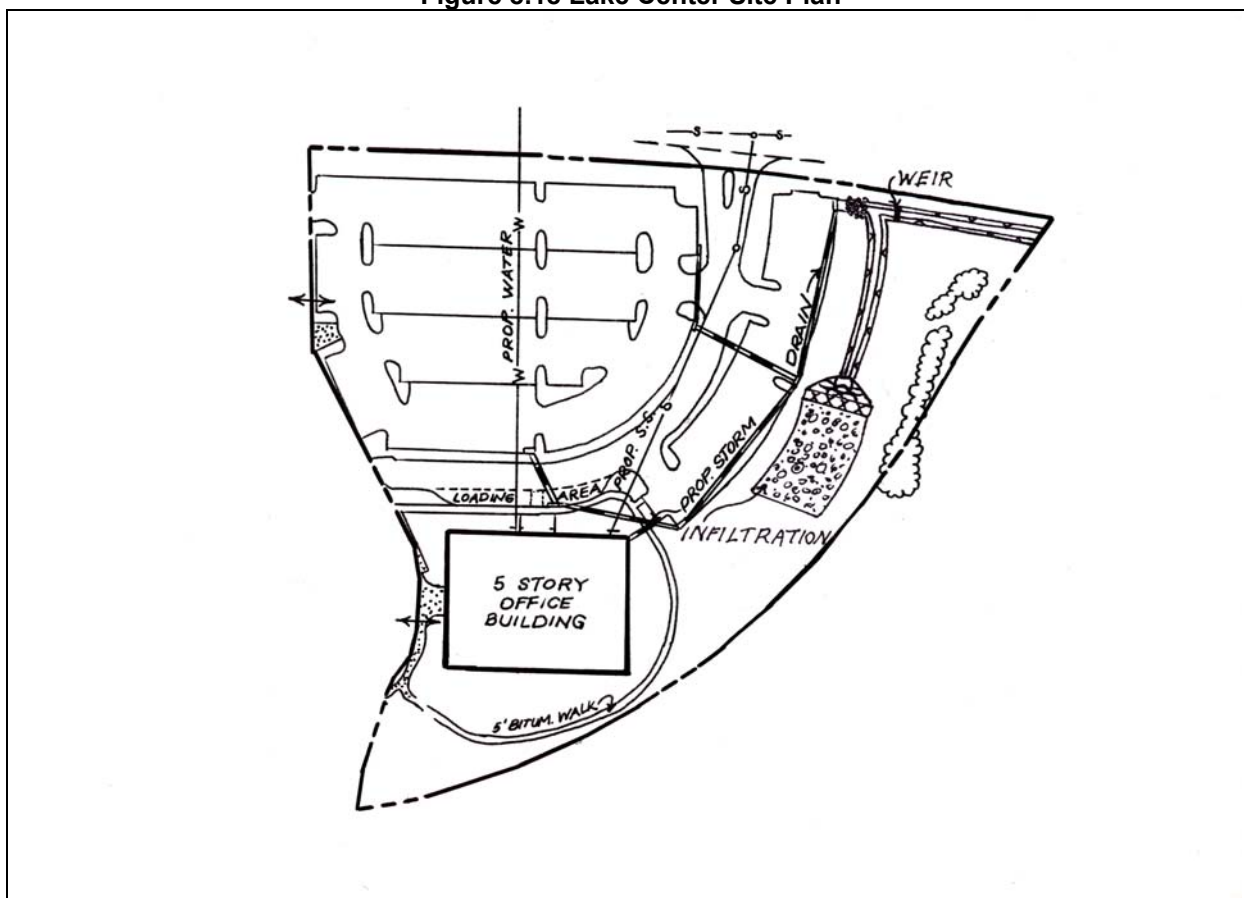
Figure 8.17 Plan and Profile of Surface Sand Filter



Section 8.4 Infiltration Trench Design Example

This design example focuses on the design of an infiltration trench for a 4.5-acre catchment of the Lake Center, a hypothetical commercial site located in Albany, NY. A five-story office building and associated parking are proposed within this catchment. The layout is shown in Figure 8.18. The catchment has 3.05 acres of impervious cover, resulting in a site impervious cover of 68%. The pre-developed site is a mixture of forest and meadow. On-site soils are predominantly HSG “B” soils.

Figure 8.18 Lake Center Site Plan



<p><u>Base Data</u> Location: Albany, NY Site Area = Total Drainage Area (A) = 4.5 ac Impervious Area = 3.05 ac; or $I = 3.05/4.50 = 68\%$ Soils Type “B”</p>	<p><i>Hydrologic Data</i></p> <table border="1"> <thead> <tr> <th></th> <th>Pre</th> <th>Post</th> </tr> </thead> <tbody> <tr> <td>CN</td> <td>58</td> <td>83</td> </tr> <tr> <td>t_c (hrs)</td> <td>.44</td> <td>.10</td> </tr> </tbody> </table>		Pre	Post	CN	58	83	t_c (hrs)	.44	.10
	Pre	Post								
CN	58	83								
t_c (hrs)	.44	.10								

This step-by-step example will focus on meeting the water quality requirements. Channel protection control, overbank flood control, and extreme flood control are not addressed in this example. Therefore, a detailed hydrologic analysis is not presented. For an example of detailed sizing calculations, consult section 8.1. In general, the primary function of infiltration practices is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. For this example, the post-development 10-yr peak discharge is provided to appropriately size the necessary by-pass flow splitter. Where quantity control is required, bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults).

Step 1. Compute design volumes and flows using the Unified Stormwater Sizing Criteria.

Design values are presented in Table 8.4 below.

Table 8.4 Site Design Hydrology					
Condition	CN	WQ _v	Q ₁	Q ₂	Q ₁₀
		<i>ft³</i>	<i>cfs</i>	<i>cfs</i>	<i>cfs</i>
Pre-Developed	58		0.2	0.4	3
Post-Developed	83	10,781	7	10	19

Step 2. Determine if the development site and conditions are appropriate for the use of an infiltration trench.

Site Specific Data:

Table 8.5 presents site-specific data, such as soil type, percolation rate, and slope, for consideration in the design of the infiltration trench. See Appendix D for infiltration testing requirements and Appendix C for infiltration practice construction specifications.

Table 8.5 Site Specific Data	
Criteria	Value
Soil	Silt Loam
Percolation Rate	0.5"/hour
Ground Elevation at BMP	219'
Seasonally High Water Table	211'
Local Ground Slope	<1%

Step 3. Confirm local design criteria and applicability.

Table 8.6, below, summarizes the requirements that need to be met to successfully implement infiltration practices. On this site, infiltration is feasible, with restrictions on the depth and width of the trench.

Table 8.6 Infiltration Feasibility	
Criteria	Status
Infiltration rate (f_c) greater than or equal to 0.5 inches/hour.	<ul style="list-style-type: none"> Infiltration rate is 0.5 inches/hour. OK.
Soils have a clay content of less than 20% and a silt/clay content of less than 40%.	<ul style="list-style-type: none"> Silt Loam meets both criteria.
Infiltration cannot be located on slopes greater than 6% or in fill soils.	<ul style="list-style-type: none"> Slope is <1%; not fill soils. OK.
Hotspot runoff should not be infiltrated.	<ul style="list-style-type: none"> Not a hotspot land use. OK.
The bottom of the infiltration facility must be separated by at least three feet vertically from the seasonally high water table.	<ul style="list-style-type: none"> Elevation of seasonally high water table: 11' Elevation of BMP location: 19'. The difference is 8'. Thus, the trench can be up to 5' deep. OK.
Infiltration facilities must be located 100 feet horizontally from any water supply well.	<ul style="list-style-type: none"> No water supply wells nearby. OK.
Maximum contributing area generally less than 5 acres.	<ul style="list-style-type: none"> Area draining to facility is approximately 4.5 acres.
Setback 25 feet down-gradient from structures.	<ul style="list-style-type: none"> Trench edge is > 25' from all structures. OK.

Step 4. Size overflow channel.

Water flows from the edge of the parking lot to a 4' wide, flat bottom channel with 3:1 side slopes and a 2% slope. This channel also provides pretreatment (See Step 6). Use a weir to divert the water quality volume to the infiltration trench, while allowing the 10-year event to an adjacent drainage channel and the water quality storm to flow to the infiltration trench. The peak flow for the water quality storm is 4.6 cfs (see Section 8.3 for an example calculation).

Determine the depth of flow for the water quality storm using Manning's equation. (Several software packages can be used). The following assumptions are made:

Trapezoidal channel with 3:1 side slopes

4' bottom width.

S = 2%

n varies between 0.03 at 1' depth to 0.15 at 4" depth (See Appendix L and Grass Channel Fact Sheet in Chapter 5).

Determine that the water quality storm passes at $d = 0.6'$.

Size a weir to pass the 10-year peak event, less the water quality peak flow, so that:

$$Q = 19\text{cfs} - 4.6\text{ cfs} = 14.4\text{ cfs.}$$

Use a weir length, L, of 4.0'.

By rearranging the weir equation:

$$H = (Q/CL)^{2/3} = (14.4/3.1(4))^{2/3} = 1.1'$$

Size the channel to pass the 10-year event with 6" of freeboard.

Step 5. Size the infiltration trench.

The area of the trench can be determined by the following equation:

$$A = WQv/(nd)$$

Where:

- A = Surface Area
 WQ_v = Water Quality volume (ft³)
 n = Porosity
 d = Trench depth (feet)

Assume that:

- n = 0.4
 d = 5 feet

Therefore:

$$A = 10,781 \text{ ft}^3 / (0.4 \times 5) \text{ ft}$$

$$A = 5,391 \text{ ft}^2$$

The proposed location for the infiltration trench will accommodate a trench width of up to 50 feet. Therefore, the minimum length required would be:

$$L = 5,391 \text{ ft}^2 / 50 \text{ ft}$$

$$L = 108 \text{ feet, say } 110 \text{ feet}$$

Step 6. Size pretreatment.

Pass the 10-year flow event through an overflow channel.

Size pretreatment to treat 1/4 of the WQ_v. Therefore, treat $10,781 \times 0.25 = 2,695 \text{ ft}^3$.

For pretreatment, use a pea gravel filter layer with filter fabric, a plunge pool, and a grass channel.

Pea Gravel Filter

The pea gravel filter layer covers the entire trench with 2" (see Figure 8.19). Assuming a porosity of 0.32, the pretreatment volume (P_v) provided in the pea gravel filter layer is:

$$P_{v\text{filter}} = (0.32)(2\text{''})(1 \text{ ft}/12 \text{ inches})(110')(50') = 293 \text{ ft}^3$$

Plunge Pools

Use a 50 'X20' triangular plunge pool with an average two foot depth as flow is diverted to the infiltration trench.

$$P_{V_{\text{pool}}} = (50 \times 20 \text{ ft})/2 * (2 \text{ ft}) = 1,000 \text{ ft}^3$$

Grass Channel

Accounting for the pretreatment volumes provided by the pea gravel filter and plunge pool, the grass channel then needs to treat at least $(2,695 - 293 - 1,000)\text{ft}^3 = 1,402 \text{ ft}^3$

Currently stormwater flows through a 150' long channel, with parameters described under step 4. For this channel, the flow velocity of the peak flow from the water quality storm (4.6 cfs) is approximately 1.3 fps.

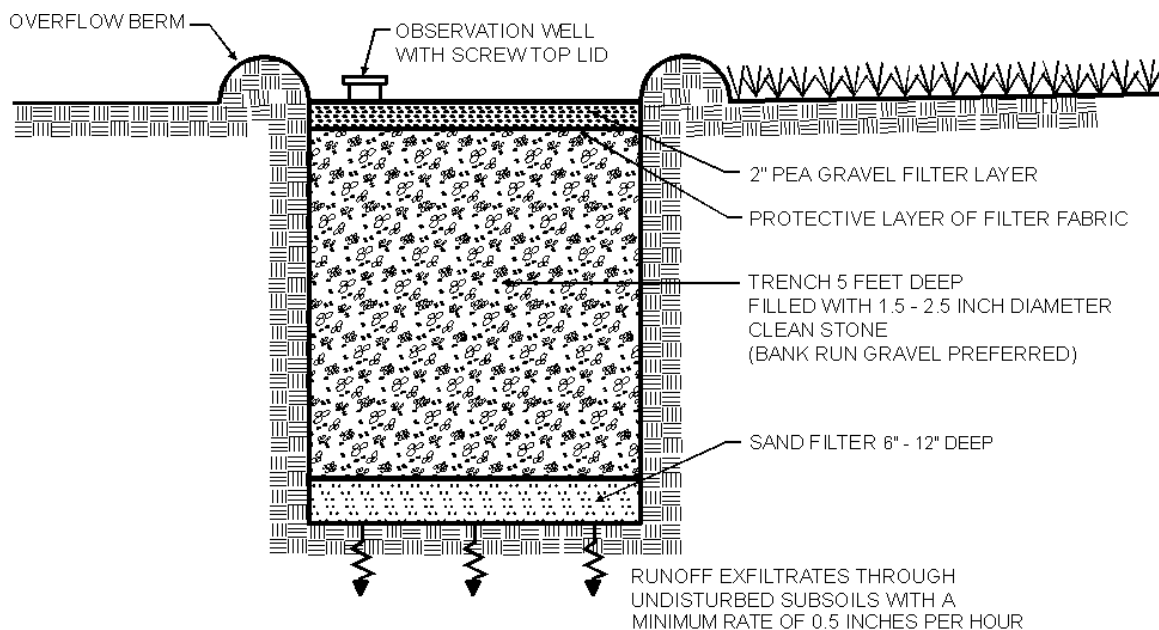
Using a required residence time of 10 minutes (600 seconds), the required length of channel for 100% of the WQ_v ($10,781 \text{ ft}^3$) would be $1.3 \text{ fps} \times 600 \text{ sec} = 780\text{ft}$.

Adjust the length to account for the volume that must be provided, or:

$$(780\text{ft}) (1,402 \text{ ft}^3)/(10,781 \text{ ft}^3) = 101 \text{ ft}$$

Therefore, for this example, a grass channel of at least 101 feet is required. 150' is OK.

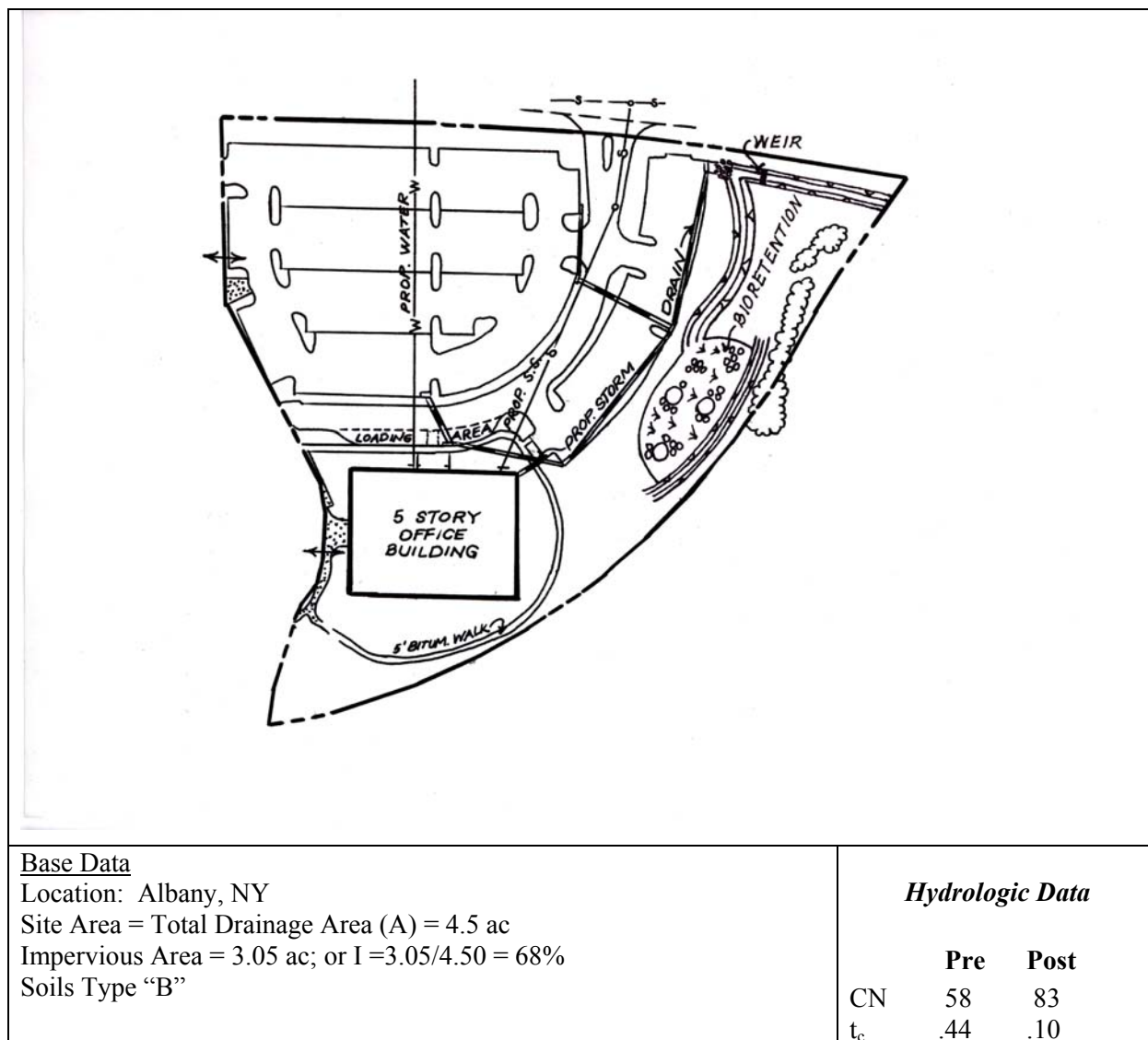
Figure 8.19 Schematic Infiltration Trench Cross Section



Section 8.5 Bioretention Design Example

This design example focuses on the design of a Bioretention area for a 4.5-acre catchment of Lake Center, a hypothetical commercial site located in Albany, NY. A five-story office building and associated parking are proposed within this catchment. The layout is shown in Figure 8.20. The catchment has 3.05 acres of impervious cover, resulting in 68% impervious cover. The pre-developed site is a mixture of forest and meadow. On-site soils are predominantly HSG “B” soils.

Figure 8.20 Lake Center Site Plan



This step-by-step example will focus on meeting the water quality requirements. Channel protection

control, overbank flood control, and extreme flood control are not addressed in this example. Therefore, a detailed hydrologic analysis is not presented. For an example of detailed sizing calculations, consult section 8.1. In general, the primary function of bioretention is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. For this example, the post-development 2-year and 10-year peaks are used to appropriately size the grass channel leading to the facility.

Step 1. Compute design volumes using the Unified Stormwater Sizing Criteria.

Design volumes are presented in Table 8.7 below.

Condition	CN	WQ _v	Q ₁	Q ₂	Q ₁₀
		<i>ft³</i>	<i>cfs</i>	<i>cfs</i>	<i>cfs</i>
Pre-developed	58		0.3	0.6	4
Post-Developed	83	10,781	9	13	26

Step 2. Determine if the development site and conditions are appropriate for the use of a bioretention area.

Site Specific Data:

Existing ground elevation at practice location is 222.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 211.0 feet and underlying soil is silt loam (ML). Adjacent channel invert is at 213 feet.

Step 3. Determine size of bioretention filter area.

$$A_f = (WQ_v) (d_f) / [(k) (h_f + d_f) (t_f)]$$

Where:

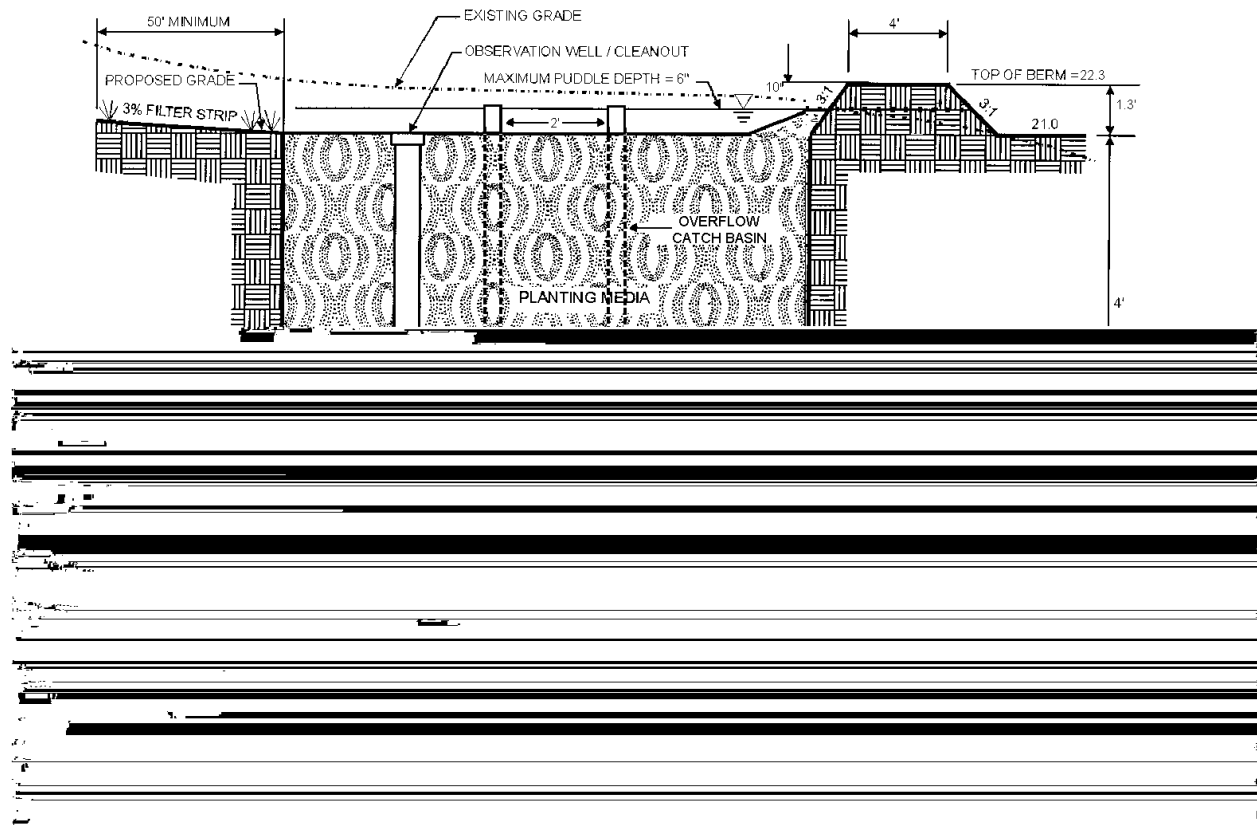
- A_f = surface area of filter bed (ft²)
- d_f = filter bed depth (ft)
- k = coefficient of permeability of filter media (ft/day)
- h_f = average height of water above filter bed (ft)
- t_f = design filter bed drain time (days) (2 days is recommended)

$$A_f = (10,781 \text{ ft}^3)(5') / [(0.5'/\text{day}) (0.25' + 5') (2 \text{ days})] \text{ (With } k = 0.5'/\text{day, } h_f = 0.25', t_f = 2 \text{ days)}$$

$$A_f = \underline{10,267 \text{ sq ft}}$$

Step 4. Set design elevations and dimensions.

Assume a roughly 2 to 1 rectangular shape. Given a filter area requirement of 10,267 sq ft, say facility is roughly 70' by 150'. Set top of facility at 219.0 feet, with the berm at 220.0 feet. The facility is 5' deep, which will allow 3' of separation distance over the seasonally high water table. See Figure 8.21 for a typical section of the facility.

Figure 8.21 Typical Section of Bioretention Facility

Step 5. Size overflow channel.

Assuming the same channel configuration as in Section 8.3, use a 4' weir set 0.63' above the base of the overflow channel. The overflow channel will flow to the adjacent drainage channel, while the water quality storm will be diverted to the bioretention cell.

Step 6. Design Pretreatment

Size pretreatment to treat $\frac{1}{4}$ of the WQ_v . Therefore, treat $10,781 \times 0.25 = 2,695 \text{ ft}^3$.

Use a grass channel to provide pretreatment. The channel has a 4' width, 2% slope and 3:1 side slopes. During the water quality event, water flows at 1.3 fps, and at a depth of 0.6' (See Section 6.3). Adjust the length to be 25% of the length required to accommodate the WQ_v for 10 minutes as follows:

$$L = (1.3 \text{ fps})(600 \text{ s})(0.25) = 195 \text{ ft.}$$

Step 7. Size underdrain area.

As a rule of thumb, the length of underdrain should be based on 10% of the A_f or 1,027 sq ft and a three-foot wide zone of influence. Using 8" perforated plastic pipes surrounded by a three-foot wide gravel bed, 10' on center (o.c.), yields the following length of pipe:

$$(1,027 \text{ sq ft})/3' \text{ per foot of underdrain} = \underline{342' \text{ of perforated underdrain}}$$

Step 8. Create overdrain design.

Size a square catch basin drop inlet to convey storms up to the peak discharge of the water quality event (4.6 cfs). Assume a 2' square, which is equivalent to an 8' weir. Rearrange the weir equation to calculate the depth of flow as follows:

$$H = [Q/(CL)]^{2/3}$$

Where,

$$Q = 4.6 \text{ cfs (flow)}$$

$$C = 3.1$$

$$H = (\text{depth of flow in feet})$$

$$L = \text{Weir Length (feet)}$$

Using this equation:

$$\begin{aligned} H &= [4.6 \text{ cfs} / (3.1)(8 \text{ ft})]^{2/3} \\ &= 0.33 \text{ feet, or } 4'' \end{aligned}$$

Allow for a 6" freeboard above the top of the catch basin. Therefore, set the elevation of the berm at 10" above the top of the catch basin.

Step 9. Choose plants for planting area.

Choose plants based on factors such as whether native or not, resistance to drought and inundation, cost, aesthetics, maintenance, etc. Select species locations (i.e., on center planting distances) so species will not "shade out" one another. Do not plant trees and shrubs with extensive root systems (e.g., willows) near pipe work. A potential plant list for this site is presented in Appendix H.